APPENDIX A ANNEXES A - D

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ANNEX A-1. HYDRAULIC DESIGN

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A.1 INTRODUCTION

The CEPP tentatively selected plan (TSP) included numerous hydraulic features throughout the entire project extent. All features were identified and summarized in the Hydraulic Design sections of the EN Appendix. Supplemental material, including further detailed design analyses, is included in this Hydraulic Design Annex. The intent of this Annex is to provide a more thorough explanation of design criteria, assumptions, and modeling analysis. Further analysis will be conducted during PED phase in order to optimize all project features for performance and cost efficiency.

A.2 LOCATION MAPS

The CEPP project components north of the redline are located north of Holey Land and STA 3/4, and bounded on the east and west by the North New River Canal and Miami Canal, respectively. All north of redline features are located within Palm Beach County. L-6 Deliveries components south of the redline lie along the L-4 and L-5 Canals, which are on the border of Palm Beach County and Broward County. Components along the L-6 Canal are located in Palm Beach County.

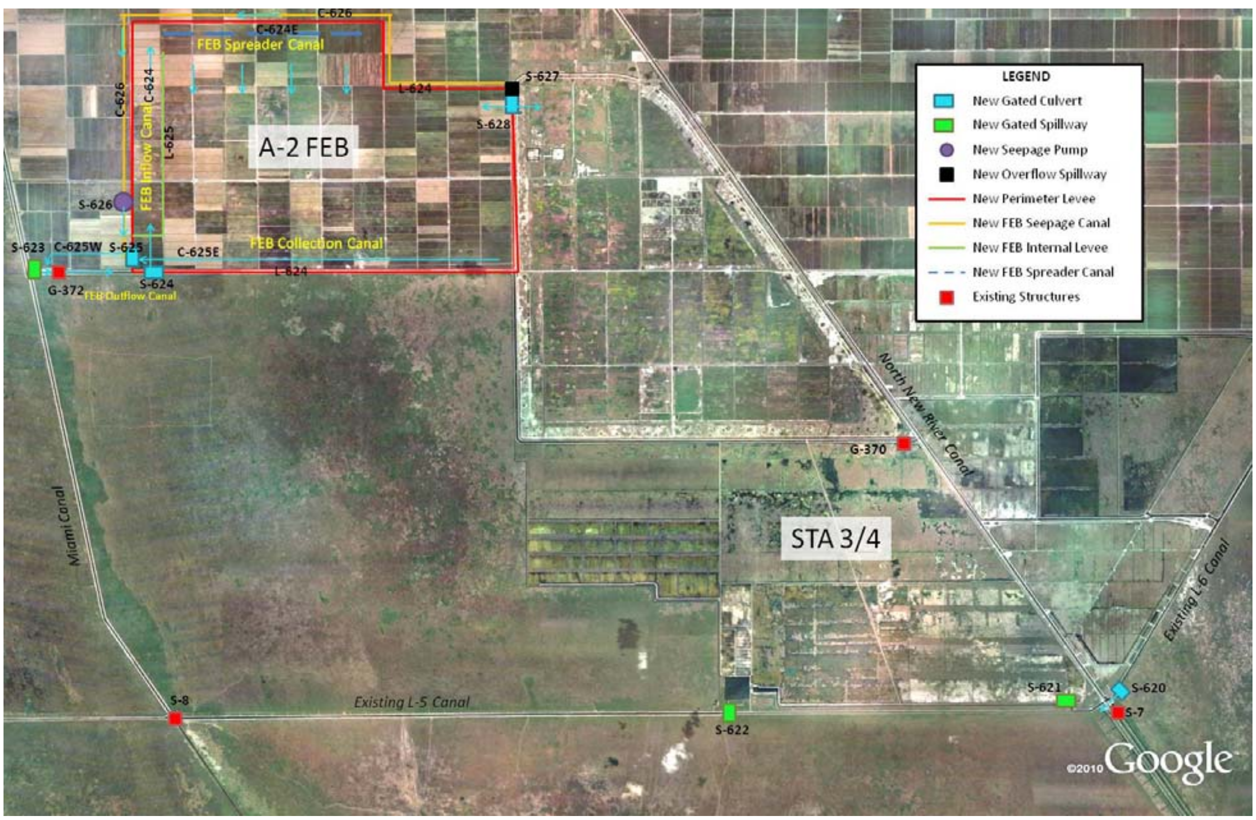


FIGURE A-1. NORTH OF THE REDLINE LOCATION MAP

FIGURE A-2. BLUE/GREEN/YELLOW LINE LOCATION MAP

A.3. NORTH OF THE REDLINE ANALYSIS

A.3.1 MIAMI CANAL IMPROVEMENTS ANALYSIS

A.3.1.1 Purpose

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing alternatives. Final design will be performed in PED phase.

A.3.1.2 Objective

Using a steady flow HEC-RAS model, determine the existing conveyance capacity of the Miami Canal reach between Lake Okeechobee and the A-2 FEB proposed location, and identify the improvements needed for the canal to convey flows from the Lake. This analysis will also identify any low-lying reaches in the levees that may need improvements based on the outlined constraints. Cross section survey data, dated 2003, was used for design work. Flow through the model ran from S-354 south to G-372, a total length of 18.5 miles.

A.3.1.3 Assumptions/Constraints

Constraints and boundary conditions were determined based on the design criteria for the S-354 and G-372 structures.

- HW stage = 12.0 ft NGVD (10.6 ft NAVD); Design TW elev at S-354[1]
- TW stage = 10.0 ft NGVD (8.6 ft NAVD); Design HW elev at G-372[2]

- Design Flow Rate = 2,000 cfs from Lake Okeechobee
- Maintain a minimum 2 ft levee freeboard

Notes:
1. Prior Correspondence with SFWMD Operations Office; ECART project modeling
2. SFWMD, Operation Plan – STA 3/4, May 2004

	Average elevation, ft NGVD	Minimum elevation, ft NGVD
East (left) levee	20.67	15.64
West (right) levee	19.09	12.29

- Manning's n: n_{bank} = 0.05, n_{canal} = 0.035 (Source: C&SF Project General Studies and Reports, Part I, Supplement 18)
- Max velocity = 2.5 fps for limestone, based on GDM for NNR Canal (November 16, 1953)
- Steepest recommended canal side slopes = 1v:1h (typical slope excavated in South Florida due to limestone, as used in Modified Waters Deliveries to ENP)

A.3.1.4 Existing Conditions

1. The existing Miami Canal conditions were analyzed from S-354 south to G-372.
    - Design flow from S-354, Q = 2,000 cfs
    - Constraint: HW = 12.0 ft NGVD (10.6 ft NAVD)
    - Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)

   Model Results – existing conditions:
    - Max WS elev = 13.03 ft NGVD (11.63 ft NAVD) *Violates max WS elev of 12.0 ft NGVD*
    - Minimum left levee freeboard = 2.61 ft
    - Minimum right levee freeboard = 1.87 ft *Violates minimum 2 ft freeboard*
    - Max channel velocity = 1.50 fps
2. The maximum conveyance through the Miami Canal with the given constraints is 1,550 cfs
    - Constraint: HW = 12.0 ft NGVD (10.6 ft NAVD)
    - Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)

   Model Results at Q=1,550 cfs:
    - Max WS elev = 12.05 ft NGVD (10.65 ft NAVD)
    - Minimum left levee freeboard = 3.59 ft
    - Minimum right levee freeboard = 2.03 ft
    - Max channel velocity = 1.27 fps

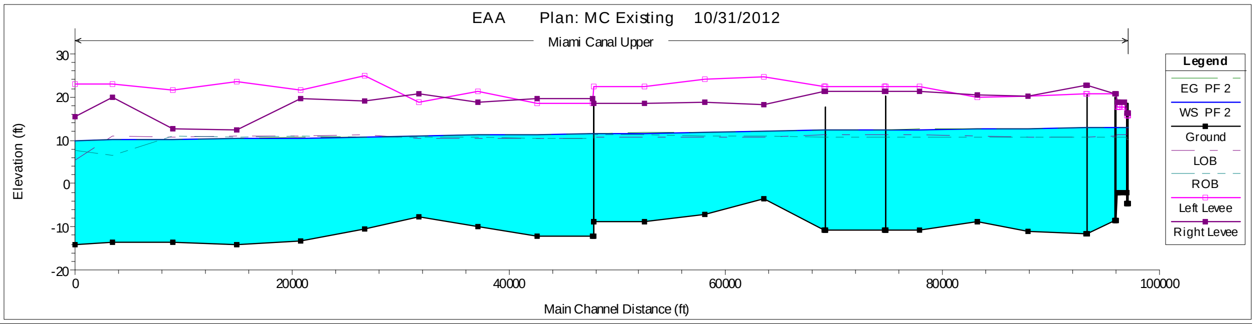


FIGURE A-3. MIAMI CANAL EXISTING CONDITIONS AT Q = 2,000 CFS (ELEVATIONS IN NGVD)

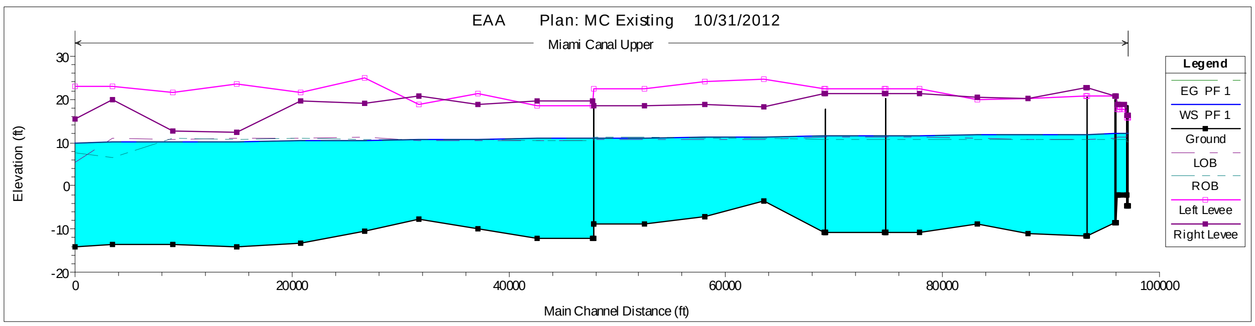
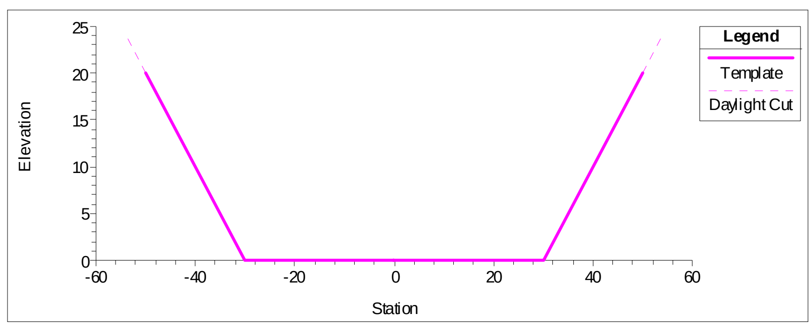


FIGURE A-4. MIAMI CANAL EXISTING CONDITIONS AT Q = 1,550 CFS (ELEVATIONS IN NGVD)

A.3.1.5 Canal Improvements

Template Design	
Template Depth:	20 ft
Bottom Width:	60 ft
Side Slope:	1
Manning's n value:	0.035
Bottom elevation:	-13.5 ft (NGVD)



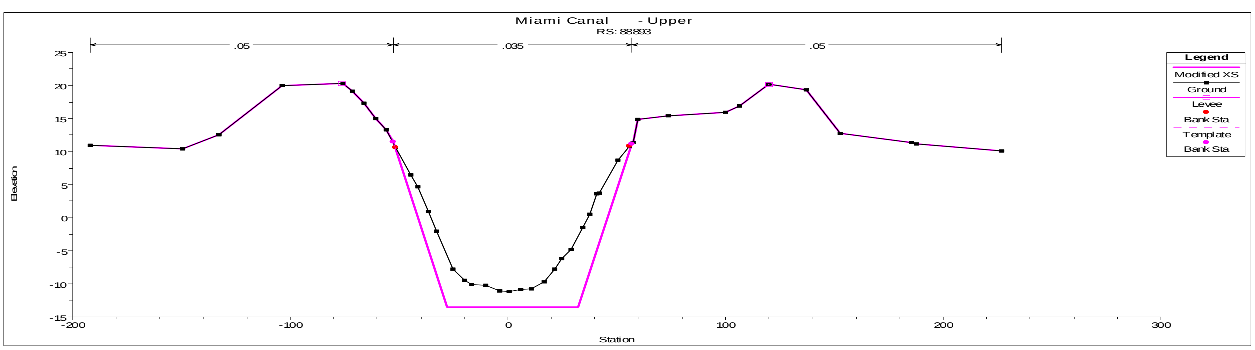


FIGURE A-5. TYPICAL CROSS SECTION OF IMPROVEMENT (ELEVATIONS IN NGVD)

Required excavation: 1,685,569 cy over a total reach length of 18.5 miles.

Model Results:

Max WS elev, ft NGVD	11.73
Max WS elev, ft NAVD	10.33
Minimum Left Levee Freeboard, ft	3.91
Minimum Right Levee Freeboard, ft	2.08
Max Channel Velocity, fps	1.7

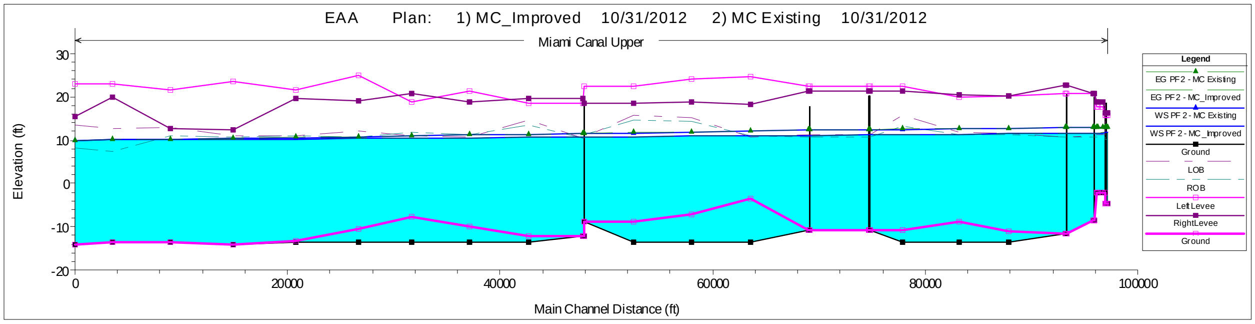


FIGURE A-6. FLOW PROFILE OF MIAMI CANAL WITH IMPROVEMENTS COMPARED TO EXISTING CONDITIONS (ELEVATIONS IN NGVD)

A.3.1.6 Conclusion

Modeling of the existing conditions of the Miami Canal indicated that the reach cannot convey the design flow of 2000 cfs with the assumed constraints. The existing conditions violate both the minimum freeboard criteria and the maximum upstream water surface elevation. Even with levee improvements at notable low spots, the maximum flow for the reach is 1550 cfs based on max upstream water surface elevation. The required improvements to convey the full design flow would require a bottom width expansion to 60 ft and a deepening to bottom elevation -13.5 ft NGVD (-14.9 ft NAVD). The total excavation required for those improvements is 1,685,569 cubic yards. The model results with the proposed improvements met all outlined assumptions and constraints. During modeling, the cross sections immediately upstream and downstream of the bridges were not included in improvements to avoid the need to improve/replace the bridges. The most narrow upstream/downstream bridge cross sections are located at RS 96898 and RS 96778; the cross section of RS 96898 is shown below to illustrate the existing bridge cross section to the proposed improved template.

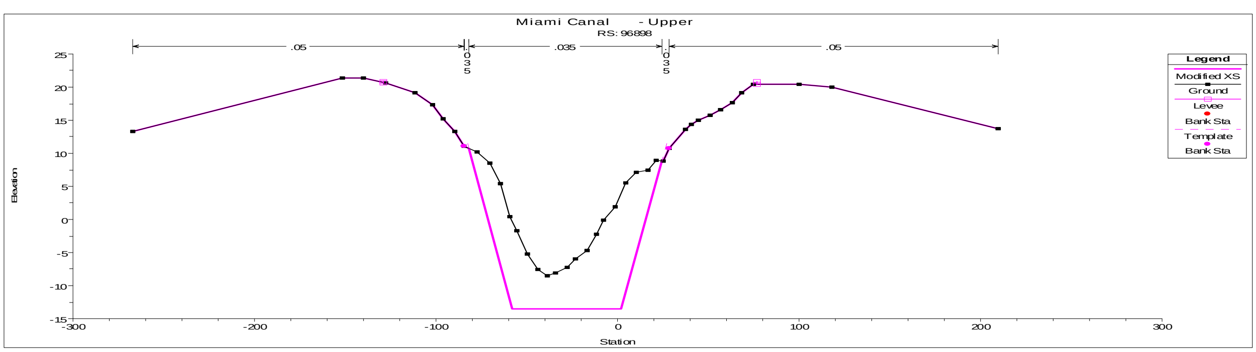


FIGURE A-7. MIAMI CANAL RS 96898 CROSS SECTION

A.3.2 NORTH NEW RIVER CANAL CONVEYANCE AND IMPROVEMENTS ANALYSIS

A.3.2.1 Purpose

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing alternatives. Final design will be performed in PED phase.

A.3.2.2 Objective

Using a steady flow HEC-RAS model, determine the existing conveyance capacity of the North New River (NNR) Canal reach between Lake Okeechobee and the A-1 FEB proposed location, and identify the improvements needed for the canal to convey flows from the Lake. Cross section survey data, dated 2003, was used for design work. Flow through the model ran from S-351 south to G-370, a total length of 22.5 miles.

A.3.2.3 Assumptions/Constraints

Constraints and boundary conditions were determined based on the design criteria for the S-351 and G-370 structures.

- HW stage = 12.0 ft NGVD (10.6 ft NAVD); Design TW elev at S-351[1]
- TW stage = 10.0 ft NGVD (8.6 ft NAVD); Design HW elev at G-370[2]
- Design Flow Rate = 2,000 cfs from Lake Okeechobee
- Maintain a minimum 2 ft levee freeboard

	Average elevation, ft NGVD	Minimum elevation, ft NGVD
East (left) levee	18.66	15.84
West (right) levee	18.56	16.34

- Manning's n: n_{bank} = 0.05, n_{canal} = 0.035 (Source: C&SF Project General Studies and Reports, Part I, Supplement 18)
- Max velocity = 2.5 fps for limestone, based on GDM for NNR Canal (November 16, 1953)
- Steepest recommended canal side slopes = 1v:1h (typical slope excavated in South Florida due to limestone, as used in Modified Waters Deliveries to ENP)

Notes:

3. Prior Correspondence with SFWMD Operations Office; ECART project modeling
4. SFWMD, Operation Plan – STA 3/4, May 2004

A.3.2.4 Existing Conditions

1. The existing NNR Canal conditions were analyzed from S-351 south to G-370.
    - Design flow from S-351, Q = 2,000 cfs
    - Constraint: HW = 12.0 ft NGVD (10.6 ft NAVD)
    - Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)

    Model Results – existing conditions:
    - Max WS elev = 13.65 ft NGVD (12.25 ft NAVD) *Violates max WS elev of 12.0 ft NGVD*

- Minimum left levee freeboard = 3.75 ft
- Minimum right levee freeboard = 4.75 ft
- Max channel velocity = 2.24 fps

2. The maximum conveyance through the NNR Canal with the given constraints is 1,350 cfs
    - Constraint: HW = 12.0 ft NGVD (10.6 ft NAVD)
    - Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)

   Model Results at Q=1,350 cfs:
    - Max WS elev = 11.99 ft NGVD (10.59 ft NAVD)
    - Minimum left levee freeboard = 4.78 ft
    - Minimum right levee freeboard = 5.78 ft
    - Max channel velocity = 1.56 fps

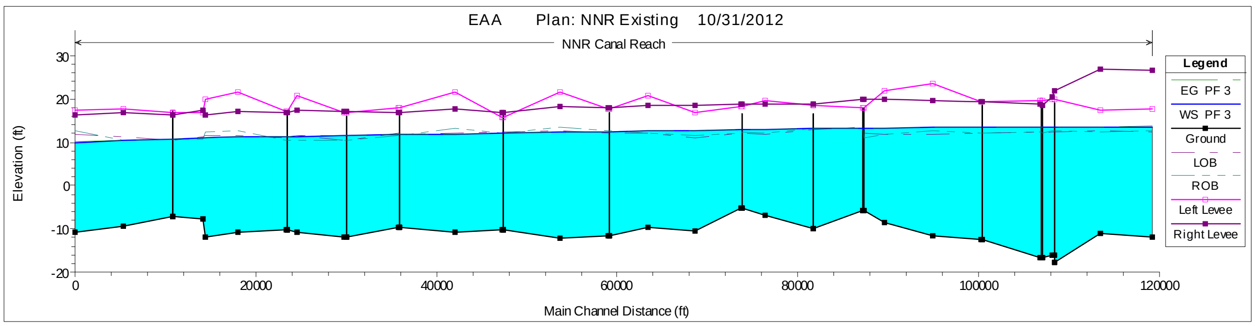


FIGURE A-8. NNR CANAL EXISTING CONDITIONS AT Q = 2,000 CFS (ELEVATIONS IN NGVD)

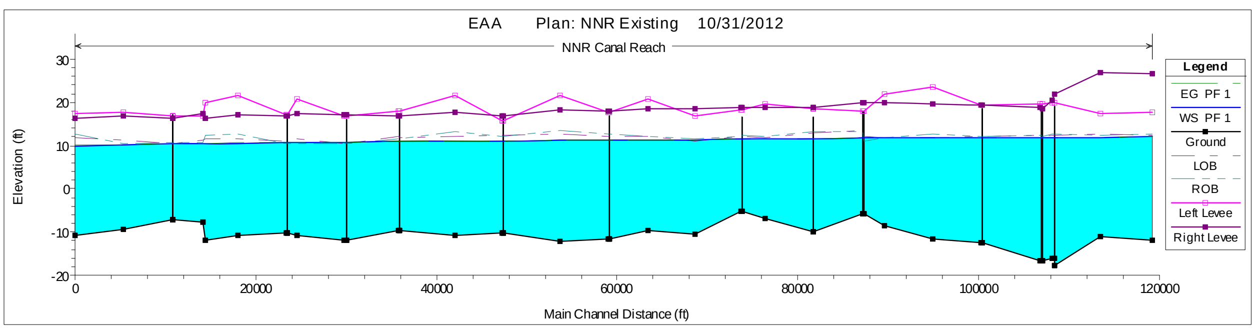
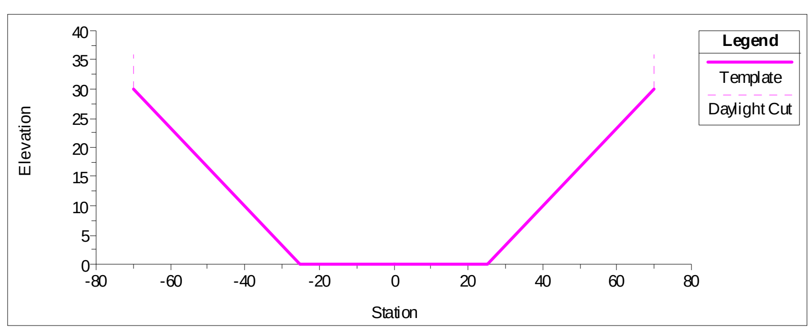


FIGURE A-9. NNR CANAL EXISTING CONDITIONS AT Q = 1,350 CFS (ELEVATIONS IN NGVD)

A.3.2.5 Canal Improvements

Template Design	
Template Depth:	30 ft
Bottom Width:	50 ft
Side Slope:	1.5
Manning's n value:	0.035
Bottom elevation:	-12.5 ft (NGVD)



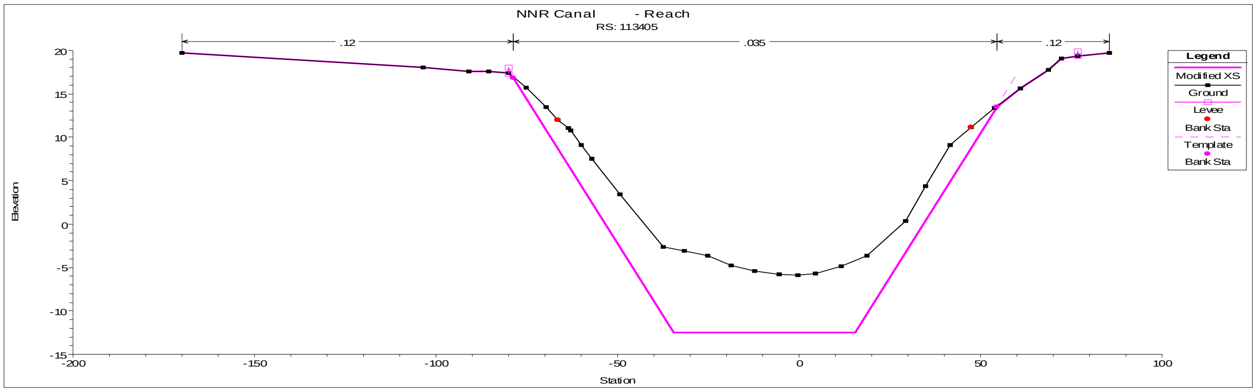


FIGURE A-10. TYPICAL CROSS SECTION OF IMPROVEMENT (ELEVATIONS IN NGVD)

Required excavation: 2,692,773 cubic yards over a total reach length of 22.5 miles.

Model Results:

Max WS elev, ft NGVD	11.97
Max WS elev, ft NAVD	10.57
Minimum Left Levee Freeboard, ft	5.04
Minimum Right Levee Freeboard, ft	6.04
Max Channel Velocity, fps	1.20

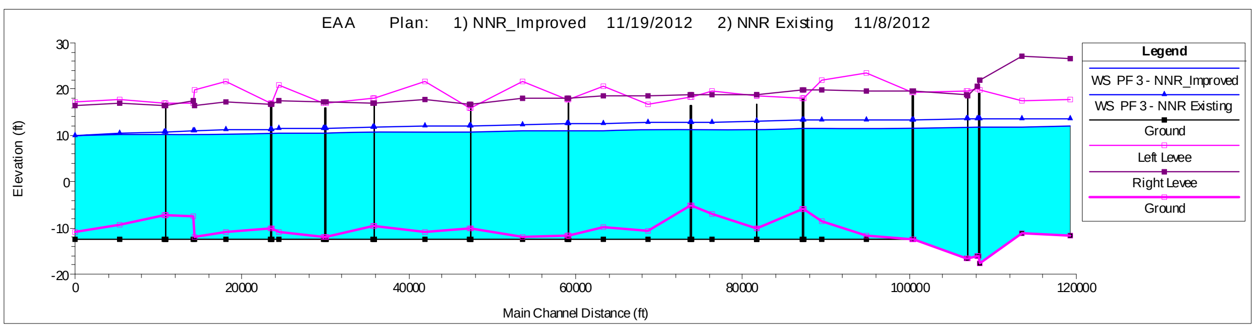


FIGURE A-11. FLOW PROFILE OF NNR CANAL WITH IMPROVEMENTS COMPARED TO EXISTING CONDITIONS (ELEVATIONS IN NGVD)

A.3.2.6 Conclusion

Modeling of the existing conditions of the NNR Canal indicated that the reach cannot convey the design flow of 2,000 cfs with the assumed constraints. The maximum flow for the reach is 1,350 cfs. The required improvements to convey the full design flow would require a bottom width expansion to 50 ft and a deepening to bottom elevation -12.5 ft NGVD (-13.9 ft NAVD). Improvements begin at RS 121023, leaving reaches in the canal where the bottom elevation is already lower than the proposed template as is. The total excavation required for those improvements is 2,692,773 cubic yards. The model results with the improvements met all outlined assumptions and constraints. Since improvements were made throughout the entire length of the NNR Canal to G-370, all bridges within the reach must be improved and/or replaced to accommodate the new channel template.

A.3.3 INFLOW CANAL ANALYSIS (CANAL C-624)

A.3.3.1 Purpose

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing alternatives. Final design will be performed in PED phase.

A.3.3.2 Objective

Using a steady-state HEC-RAS model, determine the necessary sizing of the canal providing inflow to the A-2 FEB. The canal will convey 1,550 cfs[1] from Lake Okeechobee via the Miami Canal, STA 3/4 Supply Canal, and S-624 gated control structure. The canal will be modeled as a stand-alone reach, not including geometry from the entire CEPP system; however, flows and water surface stages from upstream CEPP features will be used to establish constraints and design criteria.

A.3.3.3 Assumptions/Constraints

- Design HW stage = 14.25 ft NGVD
- Design TW stage = 13.0 ft NGVD
- Design flow rate = 1,550 cfs
- Design maximum velocity = 2.5 fps for limestone, based on GDM for NNR Canal (November 16, 1953)
- FEB perimeter levee height = 20.3 ft NGVD
- Interior levee height = 20.3 ft NGVD
- Canal length = 4.0 miles (21,120 ft)
- Manning's n: n_{bank} = 0.05, n_{canal} = 0.035 (Source: C&SF Project General Studies and Reports, Part I, Supplement 18)

A.3.3.4 Model Results

Template Design	
Template Depth	9.0 ft
Bottom Width	40 ft
Top Width	76 ft
Side Slopes (L/R)	1V:2H/1V:2H
Manning's n value	0.035
Invert elevation	0.0 ft NGVD
Bank elevation	9.0 ft NGVD
Levee elevation	20.3 ft NGVD

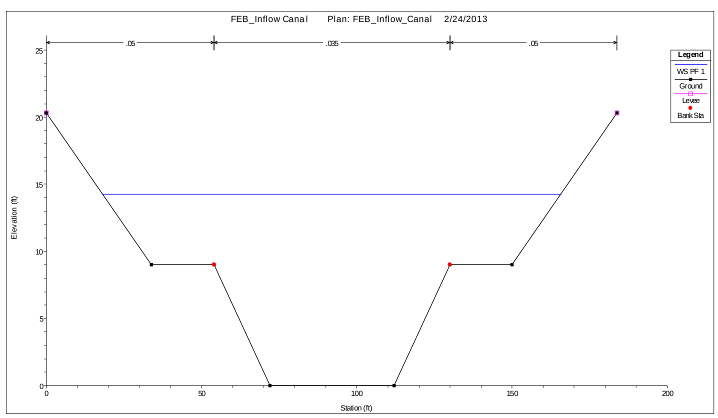


FIGURE A-12. TYPICAL CANAL CROSS SECTION (ELEVATIONS IN NGVD)

[1] Initial analysis of the Miami Canal and North New River Canal assumed 2,000 cfs discharge from Lake Okeechobee; however, the project team chose to utilize the existing capacity of the Miami Canal, therefore further design analysis used 1,500 cfs for a design flow.

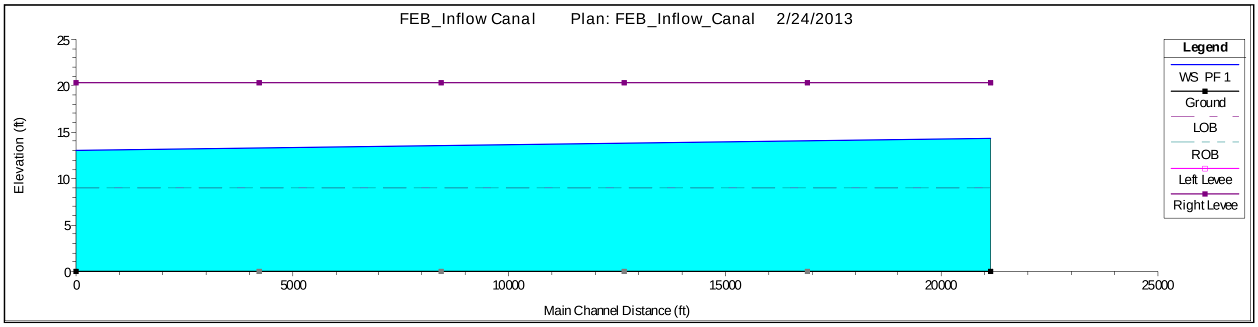


FIGURE A-13. CANAL PROFILE (ELEVATIONS IN NGVD)

River Sta	Min Ch El	W.S. Elev	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
21120	0.00	14.27	14.31	0.000049	1.51	1217.21	147.64	0.08
16896.*	0.00	14.06	14.09	0.000052	1.55	1185.56	146.5	0.08
12672.*	0.00	13.83	13.86	0.000056	1.58	1151.84	144.96	0.08
8448	0.00	13.58	13.61	0.000061	1.63	1115.70	143.46	0.08
4224.*	0.00	13.30	13.34	0.000067	1.68	1076.65	141.82	0.09
0	0.00	13.0	13.04	0.000075	1.74	1034.00	140.0	0.09

TABLE A-1. HEC-RAS OUTPUT (* INDICATES INTERPOLATED CROSS SECTIONS)

A.3.3.5 Conclusion

A steady flow HEC-RAS analysis was conducted to determine the necessary canal template to convey the design flow rate. The resulting canal design required a template with a bottom width of 40 feet, canal depth of 9.0 feet, maximum water surface depth of 14.27 feet, and a total length of 4.0 miles. The S-624 (DS-5) gated structure is at the headwater of the inflow canal, conveying flows from the Miami Canal via the STA 3/4 Supply Canal. Due to the locations of the S-624 and S-625 (DS-7) structures along the FEB perimeter, the inflow canal will begin approximately 100 feet from the southern FEB perimeter. Since this analysis is used primarily to provide costs for alternatives, the cross sectional flow area is the key design component to determine total excavation volumes. Optimization of canal design will be conducted during the PED phase for performance and cost efficiency.

A.3.4 FEB SPREADER CANAL DESIGN (CANAL C-624E)

A.3.4.1 Purpose

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing alternatives. Final design will be performed in PED phase.

A.3.4.2 Objective

Using an unsteady flow HEC-RAS model, determine the necessary sizing of a spreader canal along the northern border of the A-2 FEB to distribute identified inflows from the Miami Canal. The discharge of the flow should be evenly distributed across the entire length of the reach.

A.3.4.3 Assumptions/Constraints

- Design HW = 13.0 ft NGVD
- Design TW = 12.0 ft NGVD
- Length = 4 miles (21,120 ft)
- Manning's n: n_{bank} = 0.05, n_{canal} = 0.035 (Source: C&SF Project General Studies and Reports, Part I, Supplement 18)

A.3.4.4 Model Results

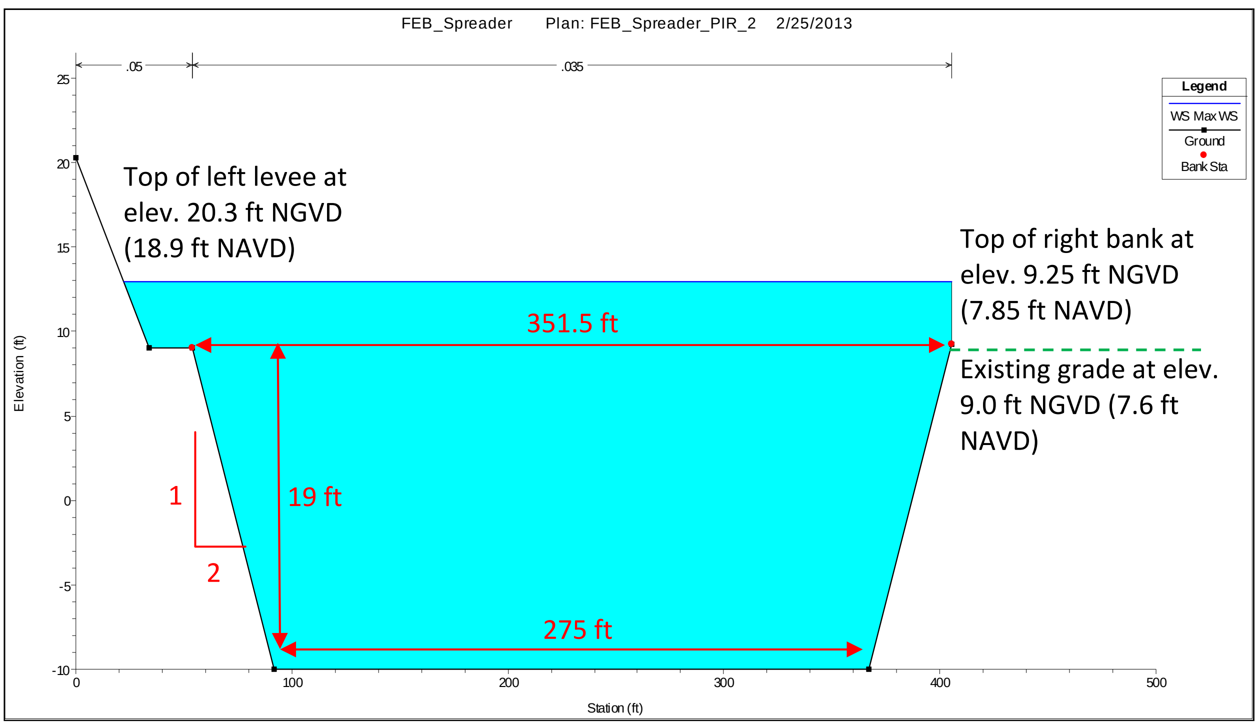


FIGURE A-14. UPSTREAM (FIRST) CROSS SECTION PROFILE (ELEVATIONS IN NGVD)

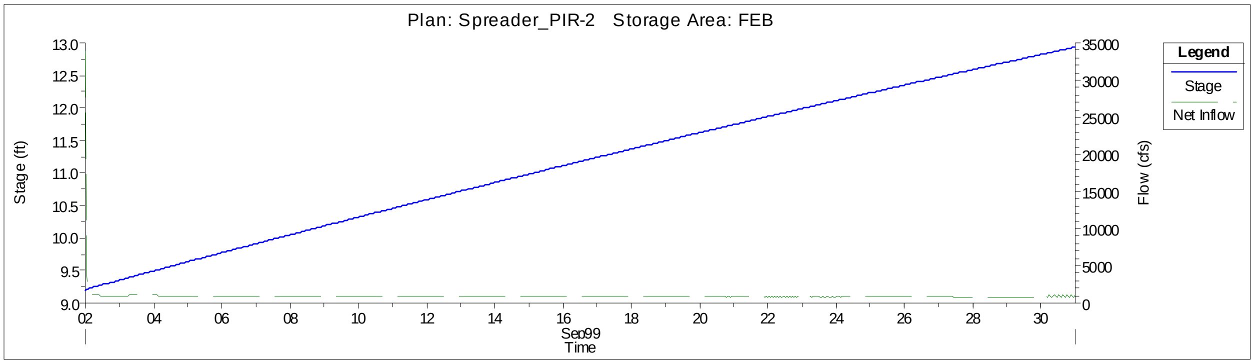


FIGURE A-15. FEB STAGE AND FLOW HYDROGRAPH (ELEVATIONS IN NGVD)

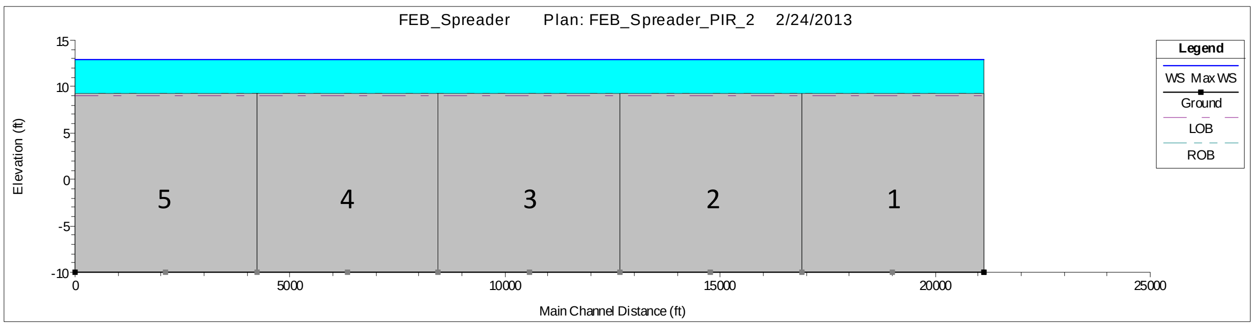


FIGURE A-16. SPREADER BERM PROFILE (ELEVATIONS IN NGVD)

Segment	Lateral Structure	Q Leaving Total (cfs)	% Total Flow
1	21119	259.06	20.68
2	16895	253.21	20.22
3	12671	249.2	19.89
4	8447	246.4	19.67
5	4223	244.81	19.54
Total		**1,252.58**	**100.00**

TABLE A-2. LATERAL STRUCTURE OUTFLOWS

A.3.4.5 Conclusion

The spreader canal was divided into five evenly spaced segments (lateral structures) of 4,224 ft in length, with a consistent top elevation at 9.25 ft NGVD; 0.25 ft above the existing ground elevation of 9.00 ft NGVD. A uniform canal template with a flow area of 5,951.75 sq ft was used throughout the length of the canal. The bottom of the canal was set at elevation -10.0 ft NGVD, with a bottom width of 275 ft. Side slopes for both the left and right banks were set to 1V:2H. The left (north) boundary will be along the FEB perimeter levee, while the right (south) bank will be a berm 0.25 ft above natural grade. This configuration produced a spreader system with evenly distributed out-of-bank flow and filled the FEB to a max stage of 12.94 ft NGVD within the 30 day simulation period (normal max pool depth at elevation 13.00 ft NGVD).

A.3.5 FEB DISCHARGE CANAL DESIGN/CANAL IMPROVEMENTS (C-625W CANAL)

A.3.5.1 Purpose

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing alternatives. Final design will be performed in PED phase.

A.3.5.2 Objective

Determine the necessary sizing of the canal providing discharge from the A-2 FEB to the headwater of the G-372 pump station. The canal will convey 1,550 cfs from the FEB through the S-625 (DS-7) gated control structure. The seepage canal adjacent to the north side of the STA 3/4 Supply Canal will be improved and used as the A-2 FEB discharge canal. The existing conditions of the seepage canal cannot accommodate the design discharge from A-2, so an expanded and deepened canal template will be required. The discharge canal will begin at S-625 and terminate upstream of the G-372, for a total length of approximately 1.5 miles. In addition to the improvements to the existing seepage canal, a short segment of a canal will have to be constructed to by-pass the G-372 seepage pump and tie in to the STA 3/4 Supply Canal. This segment is included in the total length of the reach in this analysis.

A.3.5.3 Assumptions/Constraints

- Design HW stage = 11.0 ft NGVD
- Design TW stage = 10.0 ft NGVD
- Design flow rate = 1,550 cfs
- Design maximum velocity = 2.5 fps for limestone, based on GDM for NNR Canal (November 16, 1953)
- Canal length = 1.5 miles
- Manning's n: n_{bank} = 0.05, n_{canal} = 0.035 (Source: C&SF Project General Studies and Reports, Part I, Supplement 18)

A.3.5.4 Model Results

Template Design	
Template Depth	21ft
Bottom Width	20 ft
Top Width	104 ft
Side Slopes (L/R)	1V:2H/1V:2H
Manning's n value	0.035
Invert elevation	-5.0 ft NGVD
Left bank elevation	16.0 ft NGVD
Right bank elev.	16.0 ft NGVD
Left levee elev.	23.0 ft NGVD
Right levee elev.	20.3 ft NGVD

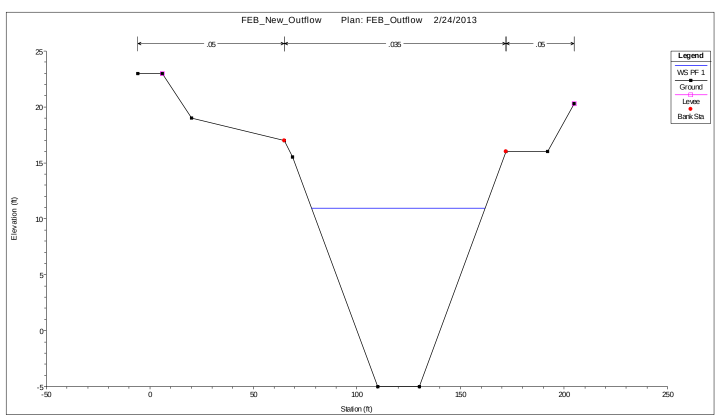


FIGURE A-17. CANAL CROSS SECTION (LOOKING DOWNSTREAM)

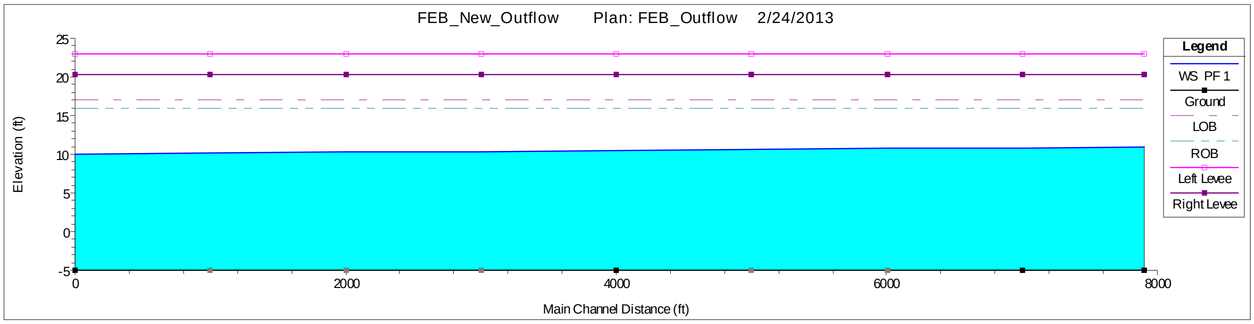


FIGURE A-18. DISCHARGE CANAL PROFILE

River Sta	Q Total	Min Ch El	W.S. Elev	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
	(cfs)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
7900	1550	-5.0	10.94	10.99	0.000103	1.87	826.89	83.76	0.11
7000	1550	-5.0	10.84	10.90	0.000106	1.89	818.92	83.38	0.11
6000*	1550	-5.0	10.73	10.79	0.000109	1.91	809.87	82.94	0.11
5000*	1550	-5.0	10.62	10.68	0.000113	1.94	800.58	82.49	0.11
4000	1550	-5.0	10.51	10.57	0.000116	1.96	791.03	82.03	0.11
3000*	1550	-5.0	10.39	10.45	0.000120	1.98	781.23	81.55	0.11
2000*	1550	-5.0	10.26	10.33	0.000125	2.01	771.13	81.05	0.11
1000*	1550	-5.0	10.13	10.20	0.000129	2.04	760.73	80.53	0.12
0	1550	-5.0	10.0	10.07	0.000134	2.07	750.00	80.00	0.12

TABLE A-3. HEC-RAS OUTPUT (* INDICATES INTERPOLATED CROSS SECTIONS)

A.3.5.5 Conclusion

The resulting canal design required a template with a bottom width of 20 feet, bottom elevation of -5.0 ft NGVD, canal depth of 21 feet, maximum water surface elevation of 10.94 ft NGVD, and a total length of 1.5 miles. Since this analysis is used primarily to provide costs for alternatives, the cross sectional flow area is the key design component to determine total excavation volumes.

A.3.6 FEB SEEPAGE COLLECTION CANAL ANALYSIS (C-626 CANAL)

A.3.6.1 Purpose

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing alternatives. Final design will be performed in PED phase.

A.3.6.2 Objective

Determine the sizing of the seepage canal necessary to accommodate the EN-GS provide seepage rate for the A-2 FEB.

A.3.6.3 Assumptions/Constraints

A seepage rate of 387 cu. ft/day/ft of levee at normal pool depth (4 ft deep) was used as provided by the Engineering Division, Soil Section (EN-GS), along with a seepage canal template and dimensions provided by the project Engineering Technical Lead (ETL). A factor of safety of 1.5 was applied to the seepage rate, giving an adjusted rate of 580.5 cu. ft/day/ft of levee. The total linear length of seepage canal around the FEB area is approximately 11 miles. The template and seepage rate were modeled in HEC-RAS to determine whether the given seepage canal geometry sufficiently conveyed the estimated seepage rate.

A.3.6.4 Conclusion

A steady flow HEC-RAS model was used to determine the necessary sizing of the seepage canal. The model determined the provided seepage canal template is sized appropriately to convey the adjusted seepage rate of 580.5 cu. ft/day/ft of levee. The maximum outflow resulted in 389.69 cfs. A seepage return pump with a total capacity of approximately 500 cfs would be required to return seepage flows back into the FEB. A proposed pump location is along the western perimeter of the FEB, north of the S-625 outlet structure and will discharge into the outflow canal. Since this analysis is used primarily to provide costs for alternatives, the cross sectional flow area is the key design component to determine total excavation volumes.

TABLE A-4. SEEPAGE COLLECTION CANAL DATA

Design seepage rate	580.5	cu. ft/day/ft of levee
	0.0067	cfs/ft of levee
Bottom Width	15.0	ft
Top Width	73.0	ft
Side Slope	1V:2H	
Average Cross sectional Flow area	507.47	sq ft
Natural grade	9.0	ft, NGVD
Bottom elevation	-5.5	ft, NGVD
Top of bank	9.0	ft, NGVD
Top of levee	18.0	ft, NGVD
Length of levee	58,000	Linear feet

	11.0	miles

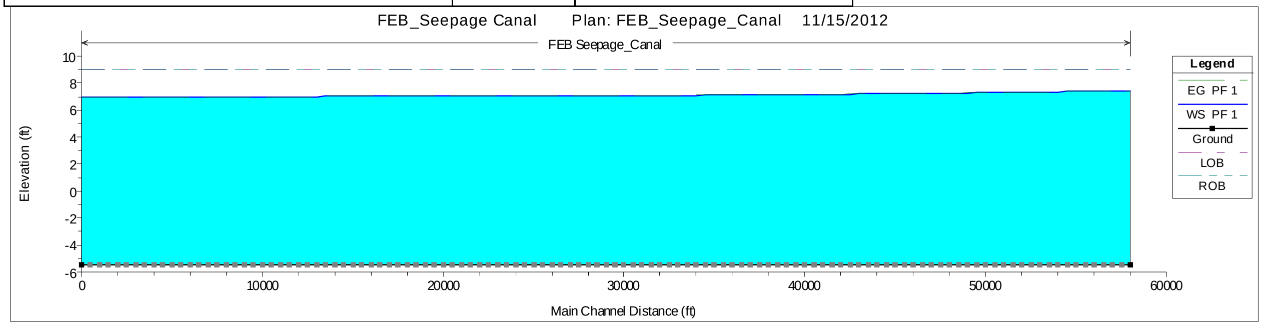


FIGURE A-19. SEEPAGE COLLECTION CANAL FLOW PROFILE

Max Water Surface Elev.	7.44	ft, NGVD
Max discharge	389.69	Cfs
Max velocity	0.74	fps

A.3.7 FEB EMERGENCY OVERFLOW SPILLWAY

A.3.7.1 Objective

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing alternatives. Final design will be performed in PED phase.

A.3.7. 2 Assumptions/Constraints

- Low hazard potential (HPC) classification; IDF = ½ PMP
- Normal pool depth = 4 ft (elev. 13.0 ft NGVD)
- Maximum pool depth = 6 ft (elev. 15.0 ft NGVD) – maintains low HPC and manages wind/wave impacts
- 50% PMP rainfall (72-hr storm) per ER-1110-8-2(FR)
- 100-yr 24hr storm per DCM-2
- 5-yr 72 hr storm per DCM-3
- Initial stage in FEB = 13.0 ft NGVD (max pool elevation)
- Downstream initial stage = 9.0 ft NGVD (natural grade)

A.3.7.3 Model Results and Analysis:

To determine the weir length, an unsteady HEC-RAS model was run comparing design criteria from ER 1110-8-2(FR) and DCMs 2 and 3. Based on DCM-1, the A-2 FEB was determined to have a low hazard potential classification (HPC). For Low HPC, DCM-2 requires the routing of the 100-yr 24-hr storm plus 60 mph wind applied to the peak surcharge stage. DCM-3 states that the Basis of Review ERP extends the basin permitted rate (storm implicit) to a 100-yr storm level to ensure that the WRDA 2000 Saving's Clause is not effectively violated. For the EAA, the ERP basin rule is 20 cfs/sq. mile (CSM) (approximately ¾" per day, or 440 cfs) for the 5-yr (assume 72-hr) storm event. Extending the discharge rate to the 100-yr 72-hr storm is above the DCM-2 requirement for low HPC impoundment/reservoir storm routing, which is the 100-yr 24-hr storm. In urban areas, the ERP rule is usually near the 20 CSM discharge rate, but it is typically combined with the 25-yr 72-hr storm event versus the 5-yr storm. Therefore, extending the ERP rule to the 100-yr 72-hr rate would provide better protection from a potential for impact than for urban areas.

Given those design criteria and guidance, the three different storm events (100-yr 24-hr; 100-yr 72-hr; and 50% 72-hr PMP) were routed with a max discharge rate of 440 cfs. The hydrographs shown in Figures A-20 to A-22 illustrate the storm rainfall and rainfall rates for each scenario. Each storm was run with a weir crest elevation of 13.00 ft NGVD and 13.50 ft NGVD; however, a final crest elevation of 13.50 ft NGVD was selected to provide an additional 6" above the normal spillway crest setting at Normal Flood Surface Level (NFSL) to prevent overtaxing of the seepage management system with more common frequent storm events since the spillway does not directly discharge into an adjacent major canal.

Table A-5 summarizes the HEC-RAS model results of each of the storm events. The use of the 100-yr 24-hr storm event at the DCM-3 recommended rate of 440 cfs resulted in a weir length of 265 feet. This allows a 3 foot freeboard on the USACE historically required 50% PMP surcharge pool peak stage on the Low HPC impoundment with the proposed minimal 9 foot embankment (nearly so, 2.85 foot freeboard actually). This freeboard lowers risk of breach with extreme storm events.

TABLE A-5. OVERFLOW SPILLWAY MODEL RESULTS

Storm	Discharge Criteria	ERP Flow rate (cfs)	Crest Elev. (ft, NGVD)	Crest Length (ft)	Max Stage (ft, NGVD)	Max Head, weir (ft)	Max Depth (ft)	Max Flow (cfs)
100-yr 24-hr	3/4"/day	440	13	110	14.13	1.13	5.13	437.01
			13.5	265	14.14	0.64	5.14	443.42
100-yr 72-hr	3/4"/day	440	13	70	14.54	1.54	5.54	442.92
			13.5	125	14.56	1.06	5.56	447.8
			13.5	265	14.52	1.02	5.52	815.38
Previous 50%, 72-hr PMP			13.5	1,500	15.03	1.53	6.03	3,007.42
			13.5	265	15.15	1.65	6.15	1,845.57

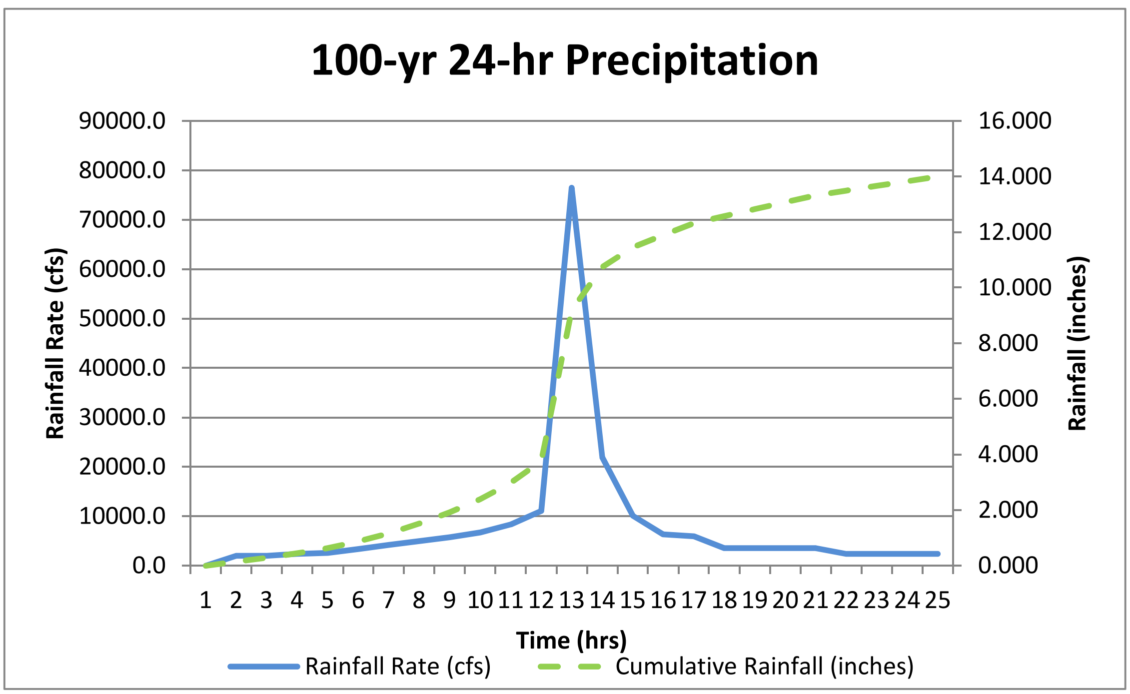


FIGURE A-20. 100-YR 24-HR STORM EVENT (Rainfall reference: SFWMD Technical Publication EMA #390, January 2001; Santa Barbara Urban Hydrograph Method (SBUH))

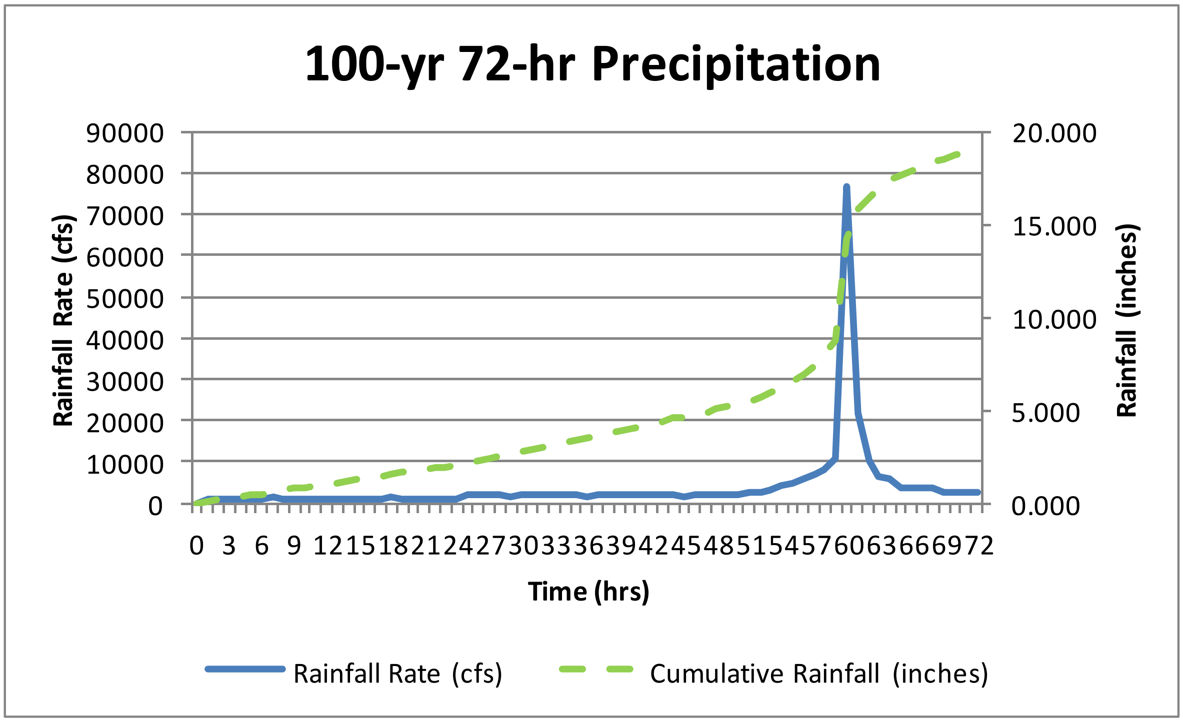


FIGURE A-21. 100-YR 72-HR STORM EVENT (Rainfall reference: SFWMD Technical Publication EMA #390, January 2001; Santa Barbara Urban Hydrograph Method (SBUH))

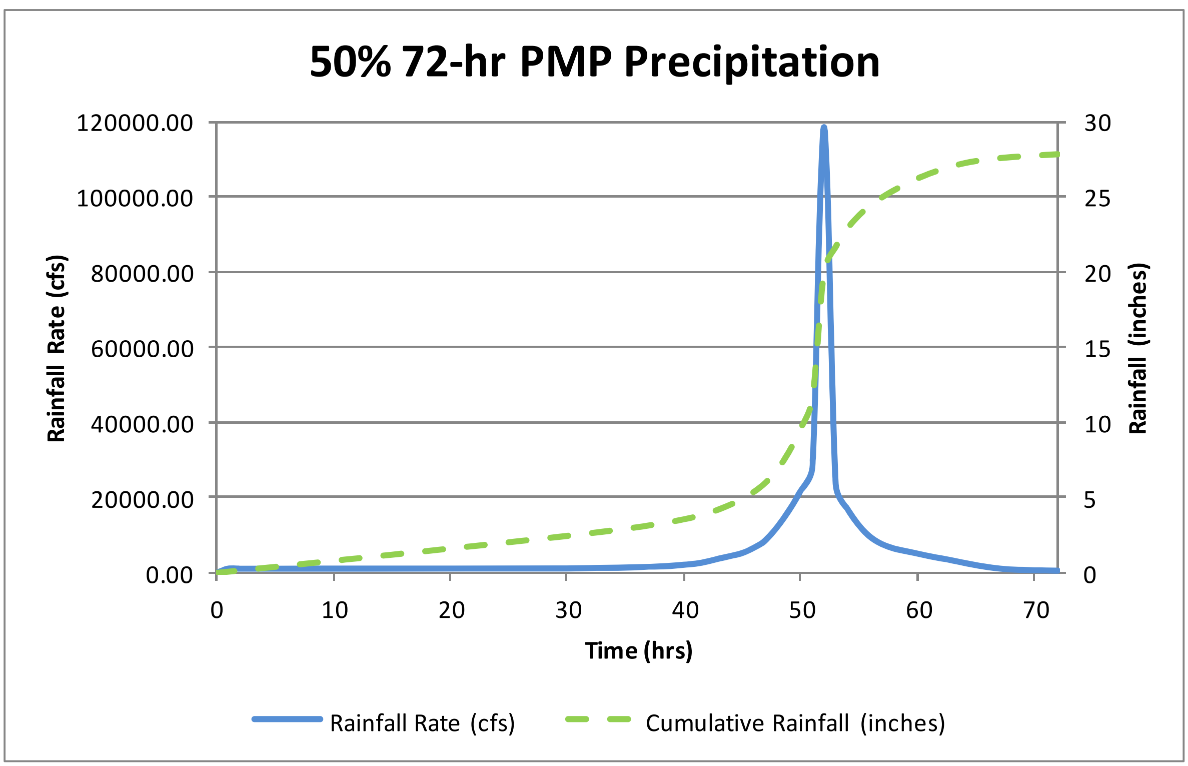


FIGURE A-22. 50% 72-HR PMP STORM EVENT (Rainfall reference: NOAA Hydrometeorological Report (HMR) 51, June 1978; Santa Barbara Urban Hydrograph Method (SBUH))

		A.3.8 S-623 Spillway								
		Miami Canal								
Structure Design Criteria										
1	**Design Discharge =**	3700	cfs				**SPF Information**			
2	**Design Headwater Elev Hd =**	10.25	ft NGVD	Hd =	6.750	feet	SPF Discharge =			cfs
3	**Analize Headwater Elev He =**	10.25	ft NGVD	He =	6.77	ft	SPF Headwater =			ft NGVD
4	**Design Tailwater Elev =**	10.00	ft NGVD	He/H'd =	1.003		SPF Tailwater =			ft NGVD
5	**Crest Elevation =**	3.50	ft NGVD	h or Hs =	6.5	ft				
6	**Single Gate Crest Width =**	35	ft	Hs/He =	0.960		**Optimum Water Surface Elevations**			
7	**Number of gates =**	4		Delta H or hd =	0.25	feet	High headwater =			ft NGVD
	Net Crest Width =	140	feet	Delta-H /He=	0.03694		High Tailwater =			ft NGVD
8	**Intermediate Pier Width =**	3.25	feet				Low Headwater =			ft NGVD
9	**Upstr Canal Bottom Width =**	100.00	feet				Low Tailwater =			ft NGVD
10	**Upstr Bottom of channel Elev =**	-13.5	ft NGVD				Max Headwater =			ft NGVD
11	**Side Slope = 1 on**	2.0	ft NGVD				Lowest tailwater =			ft NGVD
12	**Natural Grade Upstream =**	6	ft NGVD							
13	**Natural Grade Downstream =**	6	ft NGVD				**Protection Elevations**			
14	**Highest Headwater el**	12.00	ft NGVD				Wave Surge at SPF =			feet
15	**Gate clearance above water**	1.00	feet				Structure Protection Elev =			ft NGVD
	Upstream Approach Velocity =	1.06	fps				Upstream Riprip Elev =			ft NGVD
							Downstream Riprip Elev =			ft NGVD
Crest Length Reduction due to Contractions			Warning!! Check for wave runup.				BreastWall Elevation =			ft NGVD
	From Plate 7 EM 1110-2-1603		Computed Ka and Kp from charts				Clearance Elevation =			ft NGVD
16	Pier Type (1, 2, 3 or 4)	2	Kp=	0.013						
	Number of Gates =	4	Ka=	0.174						
	Number of Piers =	3								
	Width of Gates =	35	feet							
	Height of Gates	9.5	feet	Recommended height						
17	Height of Gates	14	feet	***Designer's choice***			Area (h*L) =	910	sqft	
	Gate Aspect Ratio (about 2.0) =	2.50	OK				Unit Q q =	26.429	cfs/ft	
	Top of Gate elev	13.00	ft NGVD	Clearance Elev			Upstr Depth	23.75	feet	
	L=L'-2(N *Kp+Ka)He	137.12	feet				Upstr Avg Area	3503.13	sq ft	
	Crest discharge/foot q=	26.985	cfs/ft							
	Apron Width =	149.75	feet	Net Crest Width + Pier width(s)			OR Levee Elevation			
COEFFICIENT OF DISCHARGE COMPUTATION										
18	**Trial Upstr Apron Elev =**	-2	ft NGVD							
	Computed Free Discharge Coefficients		High or Low Ogee Weir?							
	Approach Apron Height P=	5.50	feet	Ratio P/Hd =	0.815	Apron Elevation Ok				
	Approach velocity =	2.02	fps							
	Coefficient of Free Discharge Cf =	3.916	<<<<<	**From Plate 31 EM 1110-2-1603**						
19	**Designer Discharge Coeff=**	3.8500	***Designers Judgement***							
	Free Discharge Qf= C*L*He^1.5	9,293	cfs - HDC 111-4/1; Is Hs/Hc < 0.4?			**->NO!, Must Use Submerged Discharge Qs**				
DISCHARGE REDUCTION FOR SUBMERGENCE FORMULAS OUTPUT										
Low Ogee Crests Discharge Coeficient Reduction: Submerged Flow										
	From Plate 33 EM 1110-2-1603									
20	**Trial Downstr Apron Elev =**	-2.00	ft NGVD							
	Corps Reduction Factor Data d=	12.00	feet	(Hd+d)/He =	1.81	H/He =	0.04			
21	**Corps % Reduction =**	48.14%	**<<<<Look up on EM 1110-2-1603 Plate 3-5 or HDC 111-4**							
				Coefficient						
	Corps Reduced Coefficient =	0.5186	x Cf =	2.031						
	USGS Reduced Coefficient Cs/C=	0.4068	x Cf =	1.566						
	SCS Reduced Coefficient qs/q =	0.2670								
DISCHARGE REDUCED FOR SUBMERGENCE										
	REQUIRED Discharge =		3,700	cfs (From original input)						
	Corps Qs = C x (% Reduction) x L x He^1.5 =		4,820	cfs *Warning! Assumed Apron Elev, Recheck after downstream Apron design.						
				Be sure to check Apron design and Re-enter Dwnstr Elev if Reqd.						
	USDWC Qs =Cs*L*He^1.5		3,780	cfs						
	SCS Qs = (qs/qf)*Qf =		2,481	cfs						
	Average Discharge		3,694	cfs						
	SFWMD Qs =Qf*(1-(Hs/He)^1.5)^0.385 =		3,137	cfs						
									k	
	D'Aubusons Qs= k*A*(2g(Hw-Tw)+V^2)^0.5 =		3,209	cfs	k from M 1110-2-1605 pg 5-14				0.85	
	Delta H/He< 0.2 Ok to use D'Aubusons Q		3,021	cfs	If Del H >1.0 then k = 0.85; if Del H < 1.0 K= 0				0.8	

	Apron Design	**Alternate**	**Ogee**							
		Controlling	**Design**							
		1	2							
22	Design Discharge	3700	3700	cfs	**Designers Choice; Choose higher computed discharge**					
23	Headwater for Apron =	10.25	10.25	feet,ngvd						
24	Tailwater for Apron =	10	10	feet,ngvd	**!!!Lowest tailwater with maximum discharge**					
	Trial Apron Elevation =	-2	-2	feet,ngvd	**Chosen at beginning of design process**					
25	Design Apron Elevation =	-2	-8.5	feet,ngvd	***Designers choice to Change Check line10***					
	Congugate Depth "E" =	12.25	18.75	feet						
	$q/(E^{1.5})$ =	0.629	0.332							
	D_2/E =	0.5096	0.3801		**Computed From Congugate Depth Curve**					
	Computed D2 =	6.24	7.13	feet	(1)					
	Actual D2 (Tw El - Apron El) =	12	18.5	feet	(2)					
	Designers Choice D2 =	12	18.5	feet						
	Alternate Design 1 d/D2 =	192.24%		**OK - Controlling D2 ratio > 85%**						
	Alternate Design 2 d/D2 =		259.56%	**Ok-Controlling D2 Ratio > 85%**						
	D1/E =	0.0825	0.0415		**Computed From Congugate Depth Curve**					
	Computed D1 =	1.011	0.779	feet						
	Velocity at D1 Depth =	26.693	34.653	fps						
	Frude no. F1 =	4.678	6.920		**Jump Classified as =**		**Steady**	**jump**		
	Design Apron Elevation =	-2	-8.5	feet						
	Designed Apron Width =	149.75	149.75	feet						
	Average Apron Velocity =	2.06	1.34	fps						
	Hydraulic Jump Length No Baffles =	35.81	43.11	feet	**On Flat Floor No Baffles or Endsill EM1110-2-1603 (7-1)**					
	Length of basin with Baffles =	14.32	19.85	feet	**Lb=K*D1*F1^1.5 K=1.4**		**EM1110-2-1603**			
	Apron Length with Baffles =	20.46	28.35	feet	**Lb=K*D1*F1^1.5 K=2.0**		**EM1110-2-1603**			
	Apron Length (2.5xD2) =	30.00	46.25	feet	**Previous Recommendations**					
	Apron Length (3xD2) =	36.00	55.50	feet	**Previous Recommendations**					
25	Designer Apron Length =	36.00	55.00	feet	***Designers choice***		**Minimum Design OK**			
	Baffle Block Design									
	Baffle Block Height =	2.00	1.01	feet	**EM-1110-2-1603 Plate 7-4**					
26	Designer Block Height D1 =	2.00	1.00	feet	***Designers choice***					
27	Design Width of Block (D1) =	2.00	2.00	feet	***Designers choice***			74.88		
	Top of Baffle Elevation =	0.00	-7.50	feet,ngvd						
	Distance from Toe of Ogee to Upstream Face of First Row of Blocks									
	Distance from Toe of Ogee =	18.11	12.00	feet	EM-1110-2-1603 Plate 7-4					
	Distance from Toe of Ogee =	9.36	10.69	feet	First Row of Blocks from OgeeWeir 1.5*D2					
28	Designer Distance to 1st row =	18.00	12.00	feet	***Designers choice***					
	Distance from Toe of Ogee to Upstream face of Second Row of Blocks									
	Distance from Ogee toe=	15.36	19.94	feet	Second Row of Blocks from OgeeWeir First Row + 0.5*D2					
29	Designer Distance to 2nd Row =	15.00	20.00	feet	***Designers choice***					
End Sill Design				Edsill width =	1.00	feet				
	End Sill Height (D2/12) =	1.000	1.542							
	End Sill Height (D1/2) =	0.505	0.389			Slope =	13.889%			
	Recommended Height =	0.505	0.389							
30	Designer Sill Height =	0.50	1.00	feet	***Designers Choice***					
	End Sill Elevation	-1.5	-7.5	feet,ngvd						
	Additional Length ot basin	2.00	3.00	1 on 1 slope from apron floor to top of endsill						
	Total Apron Length from Ogee Toe =	38.00	58.00	feet						
	End Sill Froude No. =	0.11	0.06		ok					
	End Sill Velocity =	2.15	1.41	fps		Velocity < 9.0 fps, Design OK				
31	RipRap Velocity	5.00	5.00	fps	***Designers Recommendation***					

A.4. EAA FLOW EQUALIZATION BASIN EMBANKMENT HEIGHT EVALUATION

A.4.1 Introduction

The purpose of this study was to determine the minimum embankment height required for Everglades Agricultural Area (EAA) Flow Equalizing Basin (FEB) for a given set of design conditions.

Using a given design wind speed, pool depth, embankment slope, and embankment armor type, the wave run-up and over-wash rates at the embankment can be readily predicted. In the case of EAA FEB, this evaluation also includes the presence of vegetation. Vegetation has a dissipative effect on wave energy, which can significantly reduce wave heights and subsequent run-up and over-wash rates. Based on an allowable over-wash rate, the embankment height can then be determined.

The presence of permanent vegetation within the FEB basin allows for the use of the Vegetated Basin Evaluation Tool (VBET) (USACE, 2007). This tool combines wind speed, fetch, water depth, and vegetation type to predict the wave climate in the basin, Based upon the embankment type, slope of the embankment, and the allowable over-wash rate, the tool than determines the wave run-up on the interior embankment slopes and the resultant minimum embankment height.

A.4.2 Design Criteria

A.4.2.1 Wind Condition

The design wind condition is a key parameter in establishing embankment dimensions. For EAA FEB the design wind is 60mph. This wind speed was determined based on the recommended design wind speed for a Low Hazard Potential Classification (Low HPC) basin as outlined in Design Criteria Memorandum No. 2 (SFWMD, 2006).

A.4.2.2 Fetch

Fetch is defined as a distance over which the wind speed and direction are reasonably constant. Fetches fall into two categories, open-water fetches, where wave growth is limited only by the incident meteorological conditions, and restricted fetches, where wave growth is limited by a confined geometry such as that of a lake, river, bay, or reservoir. EAA FEB has a restricted fetch.

The restricted fetch methodology applies the concept of wave development in off-wind directions and considers the shape of the basin. The fetch is defined as the radial average over an arc of 24 degrees centered on the wind direction. For this study, the wind direction (for determination of the fetch-limited wind speed) is taken to be the direction corresponding to the maximum *averaged* (effective) fetch distance. This will provide the maximum design fetch for determining the maximum possible duration. Figure A-23 shows the FEB 24-degree arc, divided into 3-degree intervals. Averaging the radial lengths over each arc gives an effective fetch length of 34,990 feet (6.6 miles).

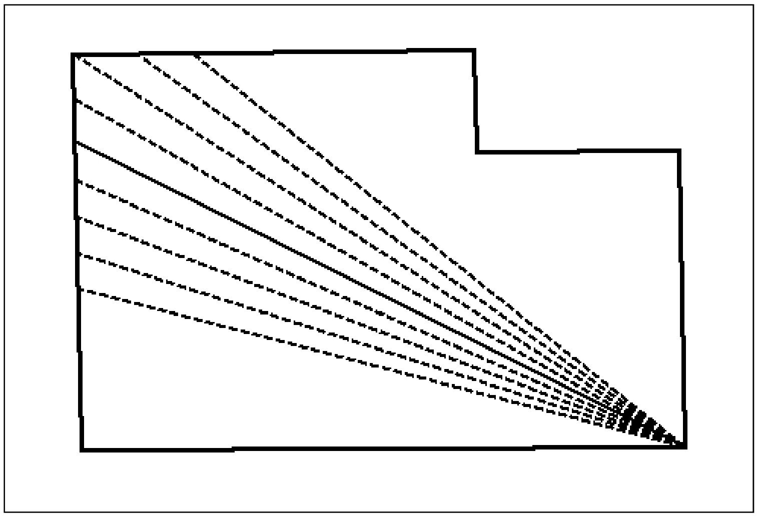


FIGURE A-23. EAA FEB FETCH DETERMINATION GRAPHIC

A.4.2.3 Water Depth

For this analysis, two water depth cases were evaluated, the 24 hour, 100 year storm maximum stage and the 50%, 72 hour PMP maximum stage. Assuming a maximum normal pool depth of 4.0 feet, the maximum water storage depths for these cases are 5.38 feet and 6.03 feet, respectively.

It should be noted that these are still water depths. Under sustained storm wind conditions, wind setup will force water levels higher at the downwind end of the basin, increasing the water depth at the toe of the embankment. For shallow basins wind setup has a significant role in determining embankment height.

Wind setup is calculated internally by the VBET model using Bretschneider's model (Ippen, 1966). Wind setup for the EAA FEB basin, under the given design conditions, is 2.3 feet for the 24 hour, 100 year storm stage case and 2.09 feet for the 50%, 72 hour PMP stage. These result in maximum total water depths at the downwind embankment of 7.68 feet and 8.09 feet, respectively.

A.4.2.4 Vegetation Type

It is assumed that vegetation, specifically cattails, will be a permanent feature of the EAA FEB basin. Therefore, embankment heights were evaluated for fully emergent vegetation. Within the VBET model, vegetation is represented by a Manning's n coefficient. Manning n values for emergent vegetation was specified as 0.35 (USACE, 1954).

A.4.2.5 Embankment Types and Slopes

In order to allow for flexibility in design, two types of embankment types were evaluated, smooth earth and riprap armored (on the interior slope). Two embankment slopes were also considered for each embankment type, 1V:3H and 1V:4H.

A.4.2.6 Allowable Over-wash Rate

Based on previous EAA documentation and on-site over-wash testing, the allowable over-wash rate was specified as 0.1 cfs/lf.

A.4.2.7 Wave Heights and Wave Periods

The VBET tool employs a look-up method to determine the wave climate within the basin. The VBET look-up database contains a series of STWAVE (Smith et al., 1999) numerical wave model runs covering a wide range of wind speed, fetch lengths, and vegetation types. Based on specified design criteria, the VBET model determines the wave height and wave period for those conditions.

A.4.3 Wave Run-up

Wave run-up can be described as the resulting forward translation of water mass that is converted from wave energy as waves encounter a sloped surface. Water rushes up the slope resulting in the vertical rise above the still water line known as run-up (Figure A-24).

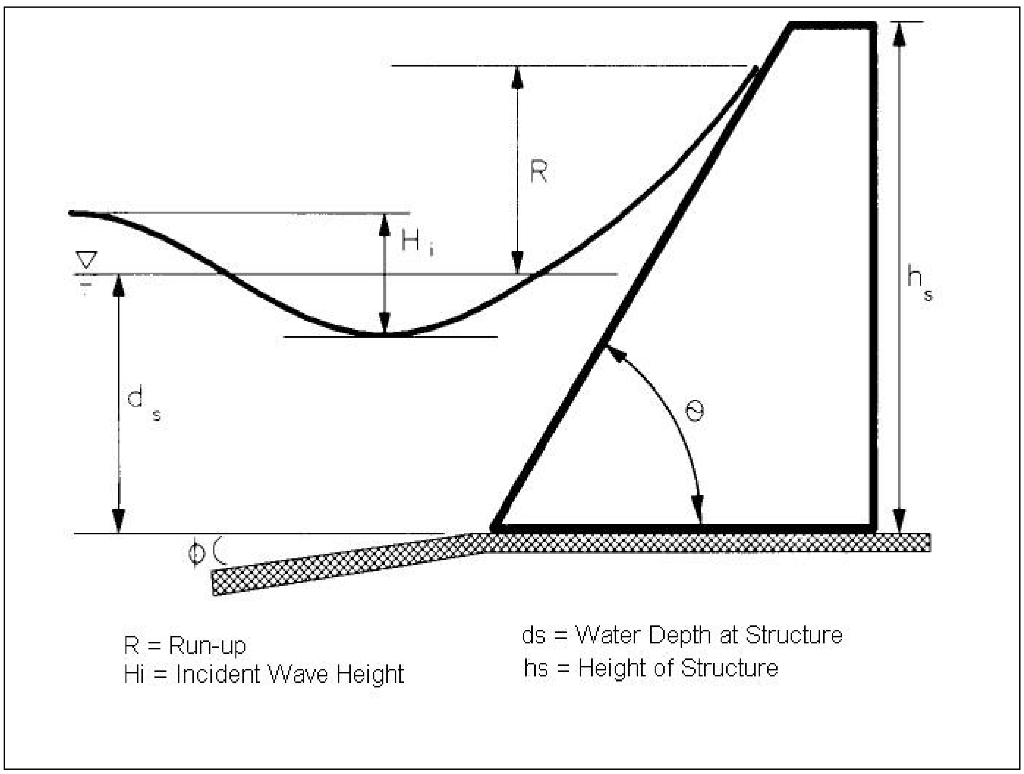


FIGURE A-24. WAVE RUN-UP DIAGRAM

A.4.3.1 Effective Depth

Although the generation of wind waves (and therefore wave run-up and over-wash) is influenced by the presence of wind setup, the relationship is highly complex and is not presently included in the methodology for determining wave height and period. Therefore, in order to include the total water level increase due to wind conditions at the down-fetch face of the levee, wind setup is generally accounted for by adding the wind setup to the design water depth at the toe of the structure. The resulting "effective depth" then becomes the water depth used during calculation of wave run-up and subsequent over-wash. The effective depths for the given design cases are 7.68 feet for the 24 hour, 100 year storm stage case and 8.09 feet for the 50%, 72 hour PMP stage.

A.4.3.2 VBET Run-up Methodologies

VBET relies on two methodologies that allow for the estimation of wave run-up on smooth (earth) and rough (riprap) embankments.

Smooth Slope Wave Run-up (Earth)
Smooth slope run-up, as presented by Ahrens and Titus (1975), recommends the following general equation:

$$R = CH_i$$

where R is wave run-up, H_i is the incident wave height, and C is a coefficient characterized by the surf similarity parameter ξ.

Rough Slope Wave Run-up (Riprap)
The rough slope wave run-up formulation, as presented by Ahrens and McCartney (1975), was empirically derived from physical model studies conducted for specific structures and wave climates. According to this method, run-up is predicted as a nonlinear function of the surf parameter, ξ, and is defined as follows:

$$R = Hi\frac{a\xi}{1+b\xi}$$

where *a* and *b* are empirical coefficients associated with slope roughness (0.956 and 0.398, respectively).

A.4.4 Over-wash

Over-wash occurs when wave run-up and wind setup levels combine to produce a water level greater than the height of the levee. Over-wash is an important design element both in terms of predicting backside flooding and safeguarding structural integrity of the levee. Several methods exist for predicting the over-wash flow rate in a given situation. The VBET model employs an irregular wave over-wash method developed by Ahrens (1977). This method uses the following assumptions: run-up values caused by an irregular wave field will follow a Rayleigh probability distribution; significant deepwater wave height, H_s, causes significant run-up R_s; and parameters α, Q^*_o, and H_o remain constant for all members of the distribution.

Ahrens estimates the over-wash rate by summing the over-wash contributions from each individual member of the run-up distribution:

$$Q = \frac{1}{199}\sum_{i=1}^{199} Q_i$$

Where Q is the volume rate of over-wash caused by irregular waves (cfs/lf) and Q_i is the volume rate of over-wash caused by one run-up on the run-up distribution.

Ahrens accounts for the effect of irregular waves when the freeboard is less than the run-up of the significant wave, R_s. When the freeboard is greater than the significant wave run-up, R_s, larger run-ups in the distribution may still overtop the structure. For these relatively high freeboards, the run-up distribution is broken into 999 elements, instead of 199, to better resolve the effect of the higher run-ups. The over-wash equation for this larger distribution becomes:

$$Q = \frac{1}{999}\sum_{i=1}^{999} Q_i$$

For a given allowable over-wash rate (to be specified by the user), the above run-up and over-wash formulations are used to determine the embankment height at which the allowable over-wash rate will not be exceeded.

A.4.5 Results

The VBET model was run for all of the design cases. Wave height and wave period at the toe of the embankment, as well as the resulting wave run-up results are shown in Table A-6.

TABLE A-6. VBET WAVE AND RUN-UP RESULTS

Design Case	Wave Height (ft)	Wave Period (sec)	Embankment Type	Embankment Slope	Wave Run-up (ft)
24hour, 100year Storm	0.08	4.3	Earth	1:3	0.20
				1:4	0.22
			Riprap	1:3	0.16
				1:4	0.15
50%, 72 hour PMP	0.10	4.3	Earth	1:3	0.24
				1:4	0.28
			Riprap	1:3	0.19
				1:4	0.18

Based on predicted wave conditions, embankment characteristics, and maximum allowable over-wash rate, a recommended embankment height can be determined. Table A-7 provides the VBET determined embankment heights with corresponding freeboards for each of the design cases.

TABLE A-7. VBET EMBANKMENT HEIGHTS

Design Case	Embankment Type	Embankment Slope	Embankment Height (ft)	VBET Freeboard* (ft)
24hour, 100year Storm	Earth	1:3	7.7	2.3
		1:4	7.7	2.3
	Riprap	1:3	7.7	2.3
		1:4	7.7	2.3
50%, 72 hour PMP	Earth	1:3	8.1	2.1
		1:4	8.1	2.1
	Riprap	1:3	8.1	2.1
		1:4	8.1	2.1
* Freeboard is defined as the vertical distance between the maximum water storage level and the embankment crest				

A.4.6 Conclusions

As shown in Table A-6, the presence of emergent vegetation damps wave energy significantly resulting in minimal wave run-up. This minimal run-up, combined with low wave energy and an allowable over-wash rate of 0.1 cfs/lf, then results in relatively low and uniform freeboard requirements for each of the design cases. In each of these cases, the VBET determined freeboard requirement is governed by the extent of wind setup rather than the amount of wave run-up. This is typical of vegetated basins with significant wave damping.

According to USACE guidance (USACE, 1991) the minimum required freeboard for a Low HPC impoundment is 3 feet above the maximum storage level. In each of the study cases, the freeboard determined by VBET (to ensure an average over-wash rate of less than 0.1cfs/lf) is less than 3 feet. Therefore, the recommended freeboard is dictated by the minimum requirement for a Low HPC basin (3.0 feet) rather than the wind and wave conditions for the site. For all variations of the 24 hour, 100 year storm case, the recommended embankment height (as measured from crest to toe) is 8.38 feet (5.38 feet maximum storage depth + 3.0 feet of freeboard). For all variations of the 50%, 72 hour PMP, the recommended embankment height is 9.03 feet (6.03 feet maximum storage depth + 3.0 feet of freeboard).

A.4.7 References

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A.5 SOUTH OF THE REDLINE ANALYSIS

A.5.1 L-5 Conveyance and Improvements Analysis

A.5.1.1 Objective

Determine the existing conveyance capacity of the L-5 Canal and identify the improvements needed for the canal portions east and west of the plug to convey flows from the L-6 and STA 3/4. Using L-5 survey data from 2007, 155 cross sections were cut 500 ft apart for a total length of 14.6 miles. Flow through the model ran from east beginning at the North New River Canal (NNR) westward to the Miami Canal.

A.5.1.2 Assumptions/Constraints

USACE and SFWMD H&H team members met via conference call on 22 August 2012 to discuss the acceptable assumptions and constraints for modeling purposes. All criteria outlined below are results from the discussion.

- HW stage = 12.0 ft NGVD (10.6 ft NAVD) at S-7 and/or G-379B
- TW stage = 10.0 ft NGVD (8.6 ft NAVD) at S-8
- Design Flow 1: Q=500 cfs at NNR from L-6 conveyance
- Design Flow 2: Q=3,000 cfs after plug [3,000 cfs = 2,500 cfs (STA 3/4) + 500 cfs (L-6), or 3,000 cfs from STA 3/4 only]
- Maintain a minimum 2 ft levee freeboard; can overtop L-5 banks as consistent with current operations

	North (right) levee average elev., ft NGVD (NAVD)	South (left) levee average elev., ft NGVD (NAVD)
East L-5	18.57 (17.17)	15.47 (14.07)
West L-5	16.82 (15.42)	22.54 (21.14)

- Manning's n: n_{bank} = 0.1, n_{canal} = 0.035 (Source: C&SF Project General Studies and Reports, Part I, Supplement 18)
- Existing plug removed and replaced with a gated spillway (refer to section A.5.1.5)
- Max velocity = 2.5 fps for limestone, based on GDM for NNR Canal (November 16, 1953).

FIGURE A-25. L-5 CANAL LOCATION MAP

Refer to Appendix A for location map and Appendix B for S-7, S-8, and G-379B exceedance plots.

A.5.1.3 Existing Conditions

A.5.1.3.1 Eastern Remnant Canal:

1. The remnant canal existing conditions were analyzed from NNR to the plug.
    - Design flow from L-6 canal, Q = 500 cfs
    - Constraint: HW = 12.0 ft NGVD (10.6 ft NAVD)
    - Boundary condition: TW = 11.5 ft NGVD (10.1 ft NAVD) – assumed a head loss of 0.5 ft for design purposes
    - Outlying high points in the channel bottom were brought to an average bottom elevation (RS=74198.19).

    Model Results:
    - Max WS elev = 12.30 ft NGVD (10.90 ft NAVD) *Violates max WS elev of 12.0 ft NGVD*
    - Minimum south (left) levee freeboard = 5.41 ft
    - Minimum north (right) levee freeboard = 2.02 ft
    - Max channel velocity = 1.22 fps

2. The maximum conveyance through the remnant canal with the given constraints is 350 cfs.
    - Constraint: HW = 12.0 ft NGVD (10.6 ft NAVD)
    - Boundary condition: TW = 11.5 ft NGVD (10.1 ft NAVD)
    - Outlying high points in the channel bottom were brought to an average bottom elevation (RS=74198.19).

    Model Results:
    - Max WS elev = 11.97 ft NGVD (10.57 ft NAVD)
    - Minimum south (left) levee freeboard = 5.82 ft
    - Minimum north (right) levee freeboard = 2.43 ft
    - Max channel velocity = 0.85 fps

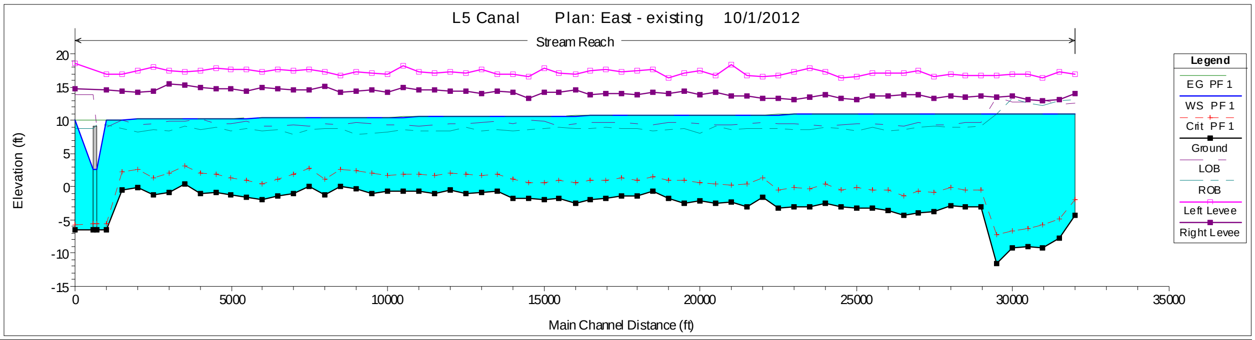


FIGURE A-26. REMNANT CANAL EXISTING CONDITIONS AT Q=500 CFS (ELEVATIONS IN NAVD)

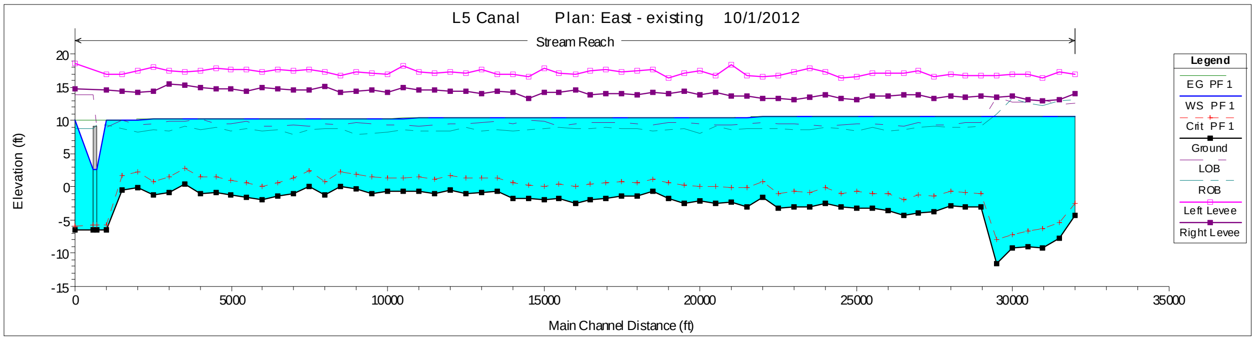


FIGURE A-27. REMNANT CANAL EXISTING CONDITIONS AT Q = 350 CFS (ELEVATIONS IN NAVD)

A.5.1.3.2 Western Canal:

1. The western canal existing conditions were analyzed from the plug to the Miami Canal.
   - Design flow from STA 3/4, Q = 3,000 cfs (assume no L-6 canal contribution)
   - Constraint: HW = 12.0 ft NGVD (10.6 ft NAVD) at G-379B (STA 3/4 outflow structure)
   - Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)
   - Outlying low points in the levees were brought to an average top elevation (RS = 45201.45 and RS = 43696.43).

   Model Results:
   - Max WS elev = 14.11 ft NGVD (12.71 NAVD) *Violates Max WS elev of 12.0 ft NGVD*
   - Minimum south (left) levee freeboard = 5.92 ft
   - Minimum north (right) levee freeboard = 2.09 ft
   - Max channel velocity = 2.55 fps

2. The maximum conveyance through the remnant canal with the given constraints is 1750 cfs.
   - Constraint: HW = 12.0 ft NGVD (10.6 ft NAVD) at G-379B (STA 3/4 outflow structure)
   - Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)
   - Outlying low points in the levees were brought to an average top elevation (RS= 45201.45 and RS = 43696.43).

   Model Results:
   - Max WS elev = 11.97 ft NGVD (10.57 ft NAVD)
   - Minimum south (left) levee freeboard = 8.06 ft
   - Minimum north (right) levee freeboard = 4.12 ft
   - Max channel velocity = 1.52 fps

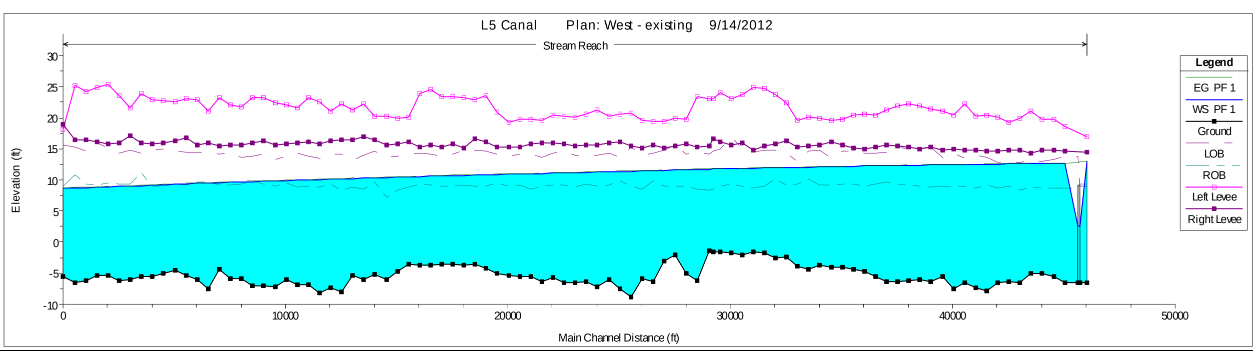


FIGURE A-28. WESTERN CANAL EXISTING CONDITIONS AT Q=3,000 CFS (ELEVATIONS IN NAVD)

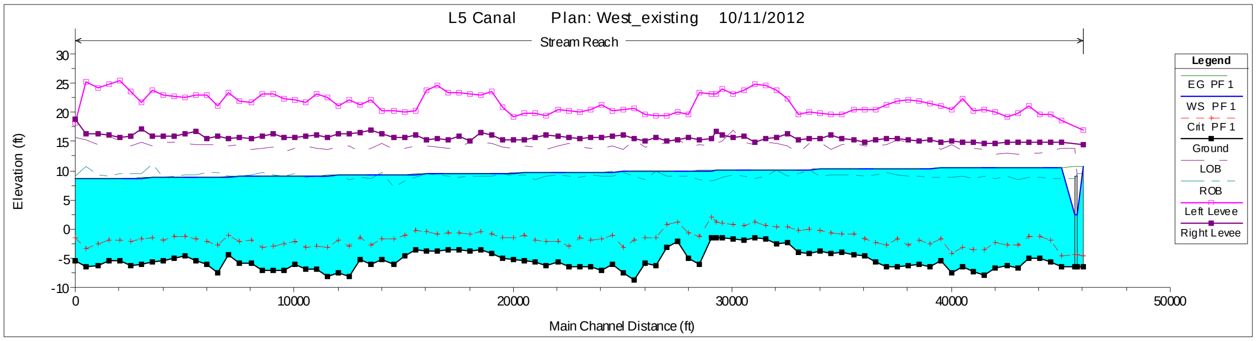


FIGURE A-29. WESTERN CANAL EXISTING CONDITIONS AT Q=1,750 CFS (ELEVATIONS IN NAVD)

A.5.1.3.3 Full L-5 Canal:

- Design flow from L-6 (500 cfs) and STA 3/4 (2500 cfs), Q = 3,000 cfs
- Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)
- Outlying low points in the levees were brought to an average top elevation (RS=74198.19).
- Outlying high points in the channel bottom were brought to an average bottom elevation (RS=45201.45 and RS = 43696.43).

<u>Model Results:</u>

- Max WS elev = 14.48 ft NGVD (13.08 ft NAVD) *Violates Max WS elev of 12.0 ft NGVD*
- Minimum south (left) levee freeboard = 3.31 ft
- Minimum north (right) levee freeboard = -0.08 ft (overtops), RS = *Violates minimum freeboard of 2.0 ft at RS = 76138.88*
- Max channel velocity = 2.55 fps

To support 500 cfs inflow from the L-6 canal via gravity conveyance structures, the conveyance in the western portion of the L-5 would need to be reduced to 1,100 cfs.

- Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)
- Outlying low points in the levees were brought to an average top elevation (RS=74198.19).
- Outlying high points in the channel bottom were brought to an average bottom elevation (RS=45201.45 and RS = 43696.43).

<u>Model Results:</u>

- Max WS elev = 12.01 ft NGVD (10.61 ft NAVD)
- Minimum south (left) levee freeboard = 5.78 ft
- Minimum north (right) levee freeboard = 2.39 ft
- Max channel velocity = 1.34 fps

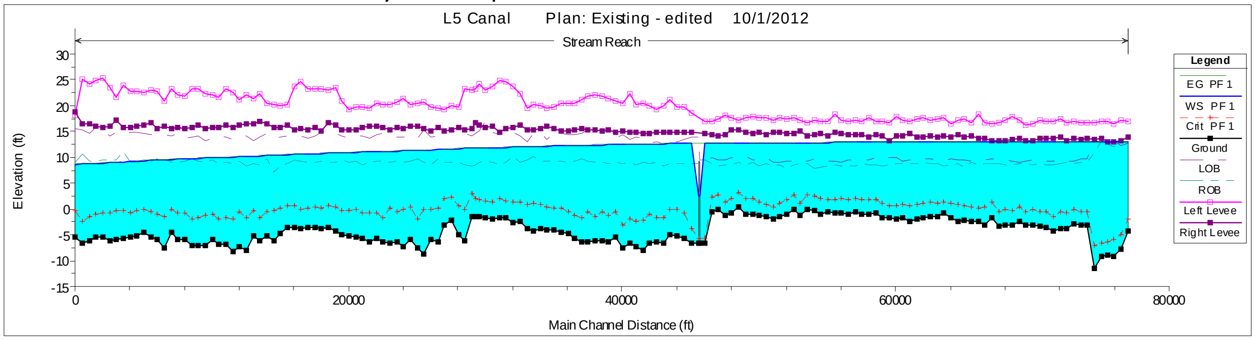


FIGURE A-30. L-5 CANAL EXISTING CONDITIONS WITH 500 CFS APPLIED AT THE UPSTREAM END AND 3,000 CFS APPLIED DOWNSTREAM OF THE PLUG REPLACEMENT (ELEVATIONS IN NAVD)

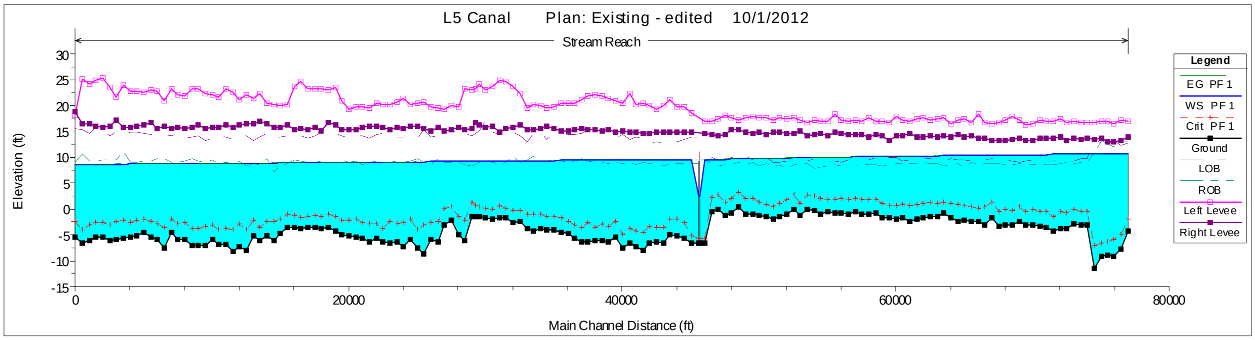


FIGURE A-31. L-5 CANAL EXISTING CONDITIONS WITH 500 CFS APPLIED AT THE UPSTREAM END AND 1,100 CFS APPLIED DOWNSTREAM OF THE PLUG REPLACEMENT (ELEVATIONS IN NAVD)

A.5.1.4 Canal Improvements

Four scenarios were considered to determine the improvements needed to the east and/or west portions of the L-5 canal to convey all proposed flows from the L-6 canal and STA 3/4; 1) full L-5 improvements and 2) western canal improvements only; 3) no L-6 conveyance; and 4) use of pump station rather than gravity structure.

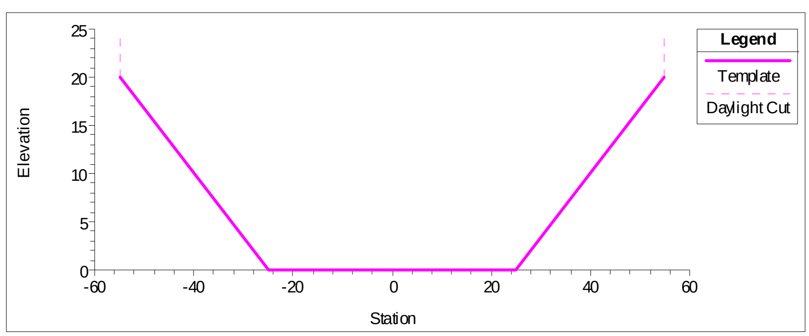
Assumptions for all scenarios:

- HW stage = 12.0 ft NGVD (10.6 ft NAVD) at S-7 and/or G-379B
- TW stage = 10.0 ft NGVD (8.6 ft NAVD) at S-8
- Q=500 cfs, simulating inflows from L-6 canal
- Q=3,000 cfs, simulating: 1) 2,500 cfs inflows from STA 3/4 plus 500 cfs inflow from L-6 canal, or 2) 3,000 cfs inflow from STA 3/4 only.
- Manning's n: n_{bank} = 0.1, n_{canal} = 0.035
- Maintain a minimum 2 ft levee freeboard; can overtop banks
- Design maximum velocity = 2.5 fps for limestone, based on GDM for NNR Canal
- Outlying low points in the levees were brought to an average top elevation.
- Outlying high points in the channel bottom were brought to an average bottom elevation.

Scenario 1: Full L-5 Canal Improvements to convey flows from L-6 and STA 3/4

East Canal:

Template Design	
Template Depth:	20 ft
Bottom Width:	50 ft
Side Slope:	1.5
Manning's n value:	0.035
Bottom elevation:	-6.5 ft (NAVD)



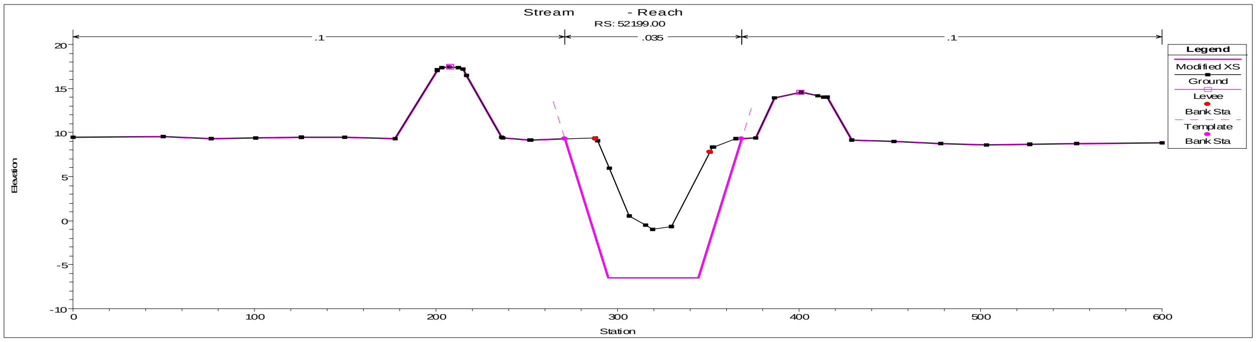
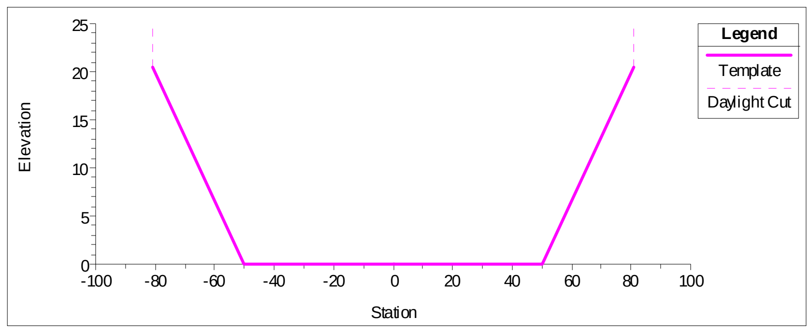


FIGURE A-32. TYPICAL CROSS SECTION OF SCENARIO 1 IMPROVEMENT; SOUTH LEVEE (LEFT), NORTH LEVEE (RIGHT) (ELEVATIONS IN NAVD)

West Canal:

Template Design	
Template Depth:	20.5 ft
Bottom Width:	100 ft
Side Slope:	1.5
Manning's n value:	0.035
Bottom elevation:	-7.0 ft (NAVD)



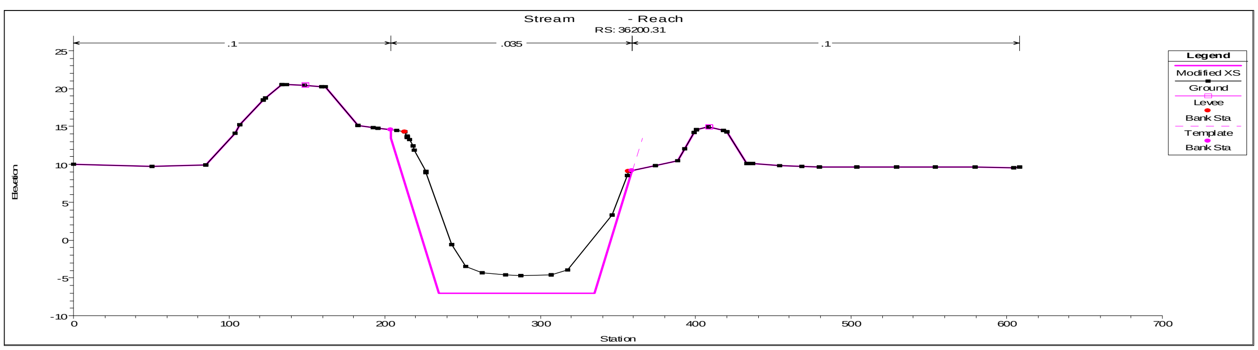


FIGURE A-33. TYPICAL CROSS SECTION OF SCENARIO 1IMPROVEMENT; SOUTH LEVEE (LEFT), NORTH LEVEE (RIGHT) (ELEVATIONS IN NAVD)

	Cut volume (cy)
Eastern Canal	732,117
Western Canal	1,242,366
TOTAL	**1,974,483**

***Note: Template depths are measured from canal bottom elevation to top of bank; not all cross sections actually measure 20 ft deep, but all have a bottom elevations of -6.5 ft NAVD and -7.0 ft NAVD, for the east and west portions respectively.**

Model Results for Scenario 1:

	West Canal	East Canal
Max WS elev (NGVD), ft	11.75	11.87
Max WS elev (NAVD), ft	10.35	10.47
Minimum Left Levee freeboard, ft	8.28	5.89
Minimum Right Levee Freeboard, ft	3.99	2.53
Max channel velocity, fps	1.56	0.39

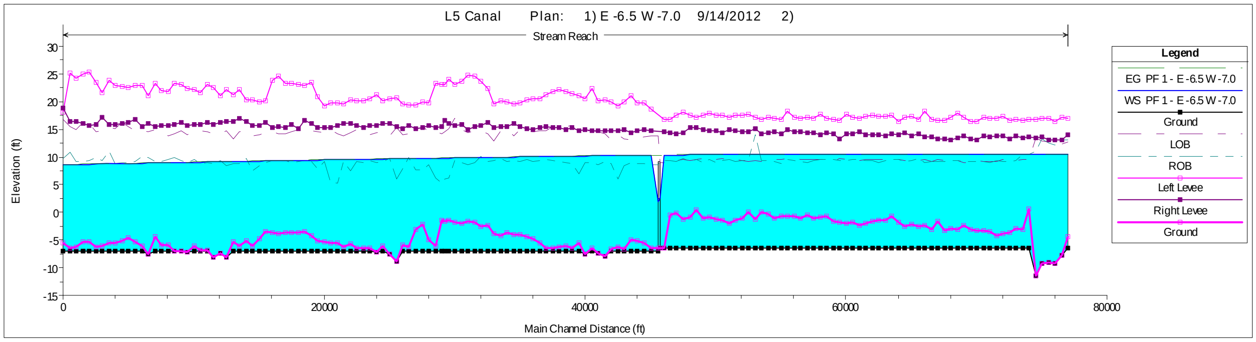
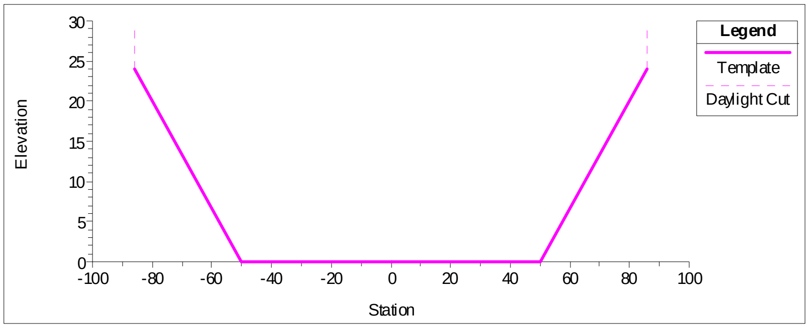


FIGURE A-34. FLOW PROFILE OF L-5 CANAL FOR SCENARIO 1 (ELEVATIONS IN NAVD)

Scenario 2: Improvements to west canal only (still assumes 500 cfs inflow from L-6)

Template Design	
Template Depth:	24 ft
Bottom Width:	100 ft
Side Slope:	1.5
Manning's n value:	0.035
Bottom elevation:	-11.0 ft (NAVD)



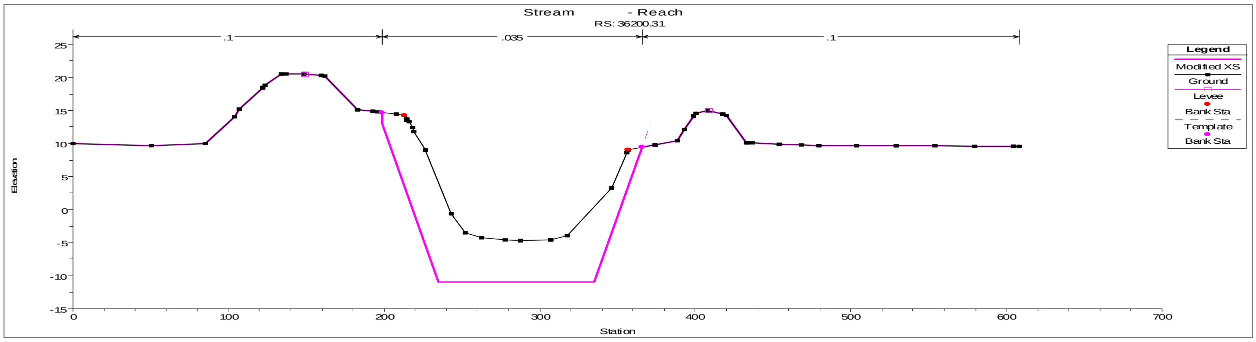


FIGURE A-35. TYPICAL CROSS SECTION OF IMPROVEMENT FOR SCENARIO 2; SOUTH LEVEE (LEFT), NORTH LEVEE (RIGHT) (ELEVATIONS IN NAVD)

	Cut volume (cy)
TOTAL	**2,298,112**

Model Results for Scenario 2:

	West Canal	East Canal
Max WS elev (NGVD), ft	10.88	12.01
Max WS elev (NAVD), ft	9.48	10.61
Minimum Left Levee freeboard, ft	9.15	5.78
Minimum Right Levee Freeboard, ft	4.83	2.39
Max channel velocity, fps	1.57	1.34

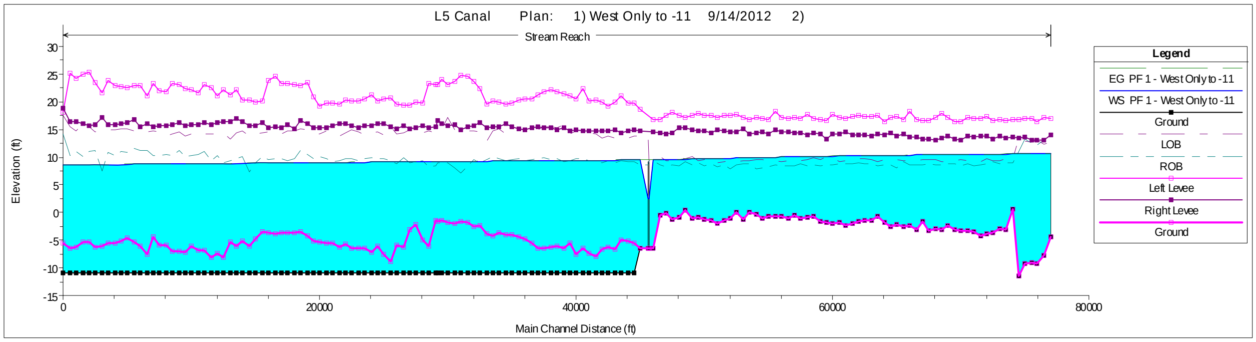
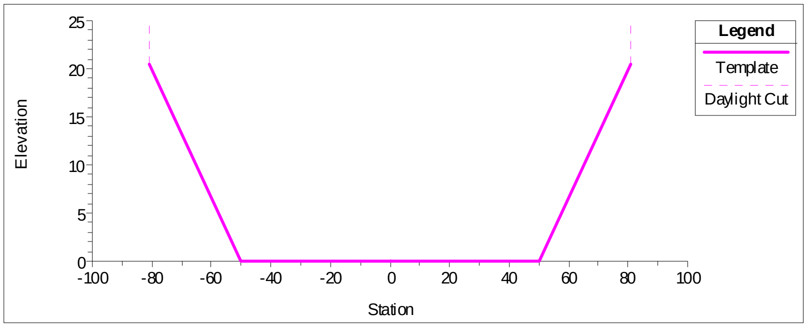


FIGURE A-36. FLOW PROFILE OF L-5 CANAL FOR SCENARIO 2 (ELEVATIONS IN NAVD)

Scenario 3: No inflow from L-6; Improvements to west canal to convey 3,000 cfs only from STA 3/4

Template Design	
Template Depth:	20.5 ft
Bottom Width:	100 ft
Side Slope:	1.5
Manning's n value:	0.035
Bottom elevation:	-7.0 ft (NAVD)



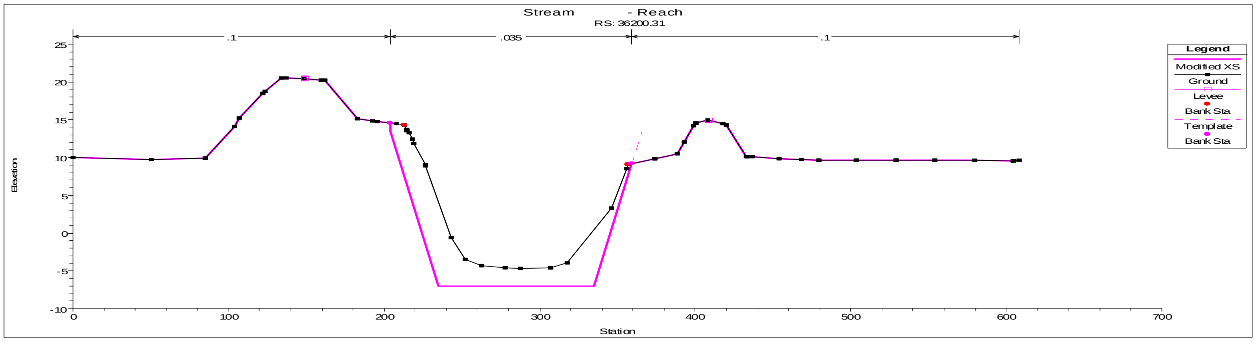


FIGURE A-37. TYPICAL CROSS SECTION OF SCENARIO 3 IMPROVEMENT (ELEVATIONS IN NAVD)

	Cut volume (cy)
TOTAL	**1,136,868**

Model Results for Scenario 3:
Boundary condition: TW = 10.0 ft NGVD (8.6 ft NAVD)

	West Canal
Max WS elev (NGVD), ft	11.59
Max WS elev (NAVD), ft	10.58
Minimum Left Levee freeboard, ft	8.05
Minimum Right Levee Freeboard, ft	3.85
.Max channel velocity, fps	1.97

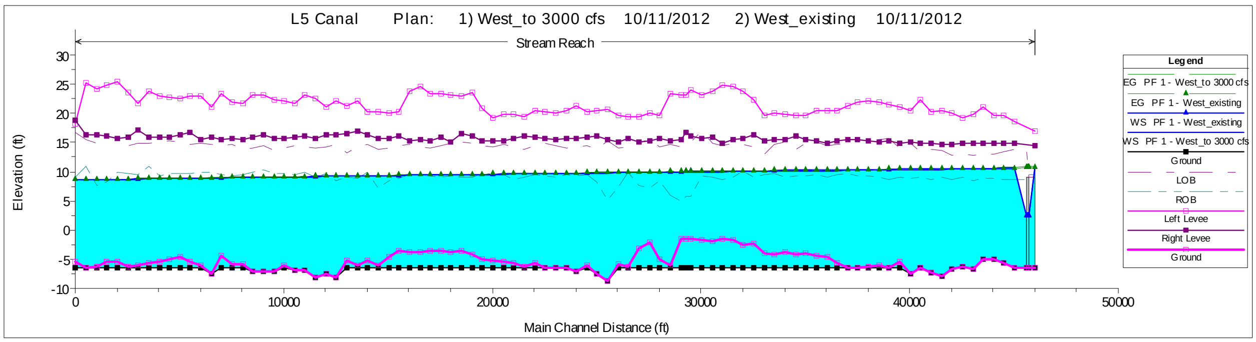


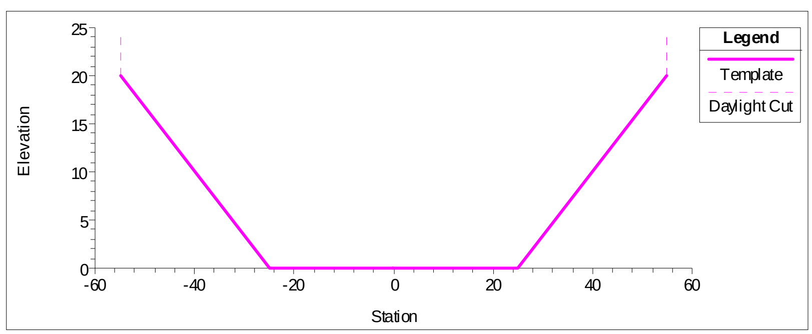
FIGURE A-38. FLOW PROFILE OF L-5 CANAL FOR SCENARIO 3 (ELEVATIONS IN NAVD)

Scenario 4: Pump station instead of gravity structure for plug replacement.

This scenario would require individual improvements of the eastern and western portions of the canal, independent of each other. The eastern portion is improved to convey 500 cfs with no back water effects from the western portion. The western portion improvements remain the same as those identified in Scenario 3.

East Canal:

Template Design	
Template Depth:	20 ft
Bottom Width:	50 ft
Side Slope:	1.5
Manning's n value:	0.035
Bottom elevation:	-1.0 ft (NAVD)



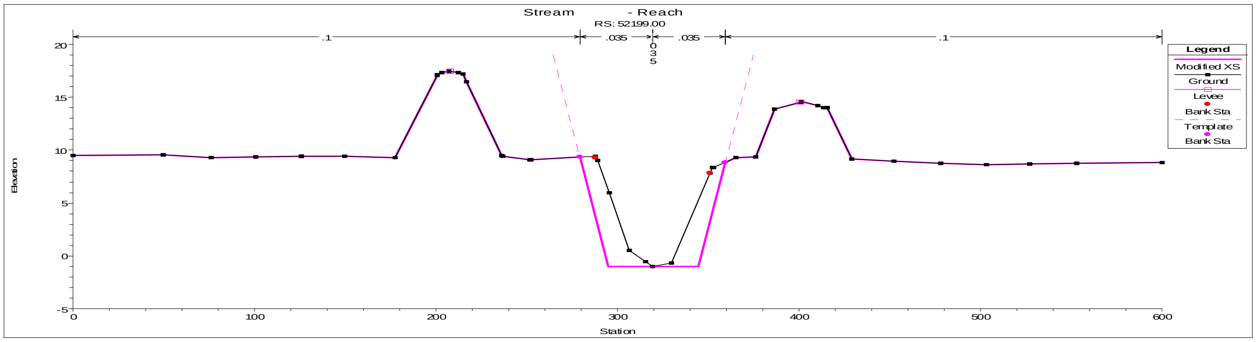


FIGURE A-39. TYPICAL CROSS SECTION OF SCENARIO 4 IMPROVEMENT (ELEVATIONS IN NAVD)

	Cut volume (cy)
Eastern Canal	233,710
Western Canal	1,136,868
TOTAL	**1,370,578**

Model Results for Scenario 4:

Boundary condition: TW = 11.5 ft NGVD (10.1 ft NAVD) – assumes 0.5 ft head loss from upstream constraint (12.0 ft NGVD)

	West Canal	East Canal
Max WS elev (NGVD), ft	11.59	11.87
Max WS elev (NAVD), ft	10.58	10.47
Minimum Left Levee freeboard, ft	8.05	5.92
Minimum Right Levee Freeboard, ft	3.85	2.53
Max channel velocity, fps	1.97	0.81

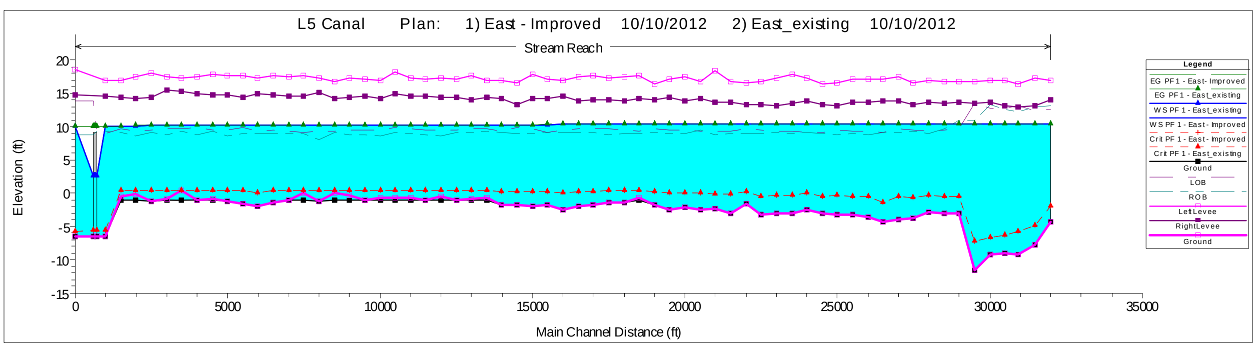


FIGURE A-40. FLOW PROFILE OF EAST L-5 CANAL FOR SCENARIO 4 (ELEVATIONS IN NAVD)

A.5.1.5 Plug Replacement

The plug separating the L-5 remnant and western canals will be fully removed and replaced with a gravity structure that will allow flow from the east (NNR, L-6 canal) to be controlled. The design aimed to minimize head loss across the structure. Preliminary designs evaluated the possibility of using box culverts; however, the resulting design capable of accommodating the proposed flows consisted of a large number and size of culverts. A gated spillway was chosen as a more suitable alternative for the plug replacement.

In order to determine HW and TW stages for the spillway, HEC-RAS analysis was conducted with L-5 Canal improvements in place and with no structure included. This scenario was run to capture the natural water surface elevations if no obstructions were present. Scenario 1 included the use of a gravity structure for the plug replacement, so the improvements resulting from that analysis were implemented for the modified geometry. The resulting water surface elevation at the cross section immediately downstream of the plug was 11.83 ft NGVD (10.43 ft NAVD). This elevation was used as the TW condition as well as a low head differential of 0.1 ft. The detailed spillway design calculations are discussed further in section A.5.2.

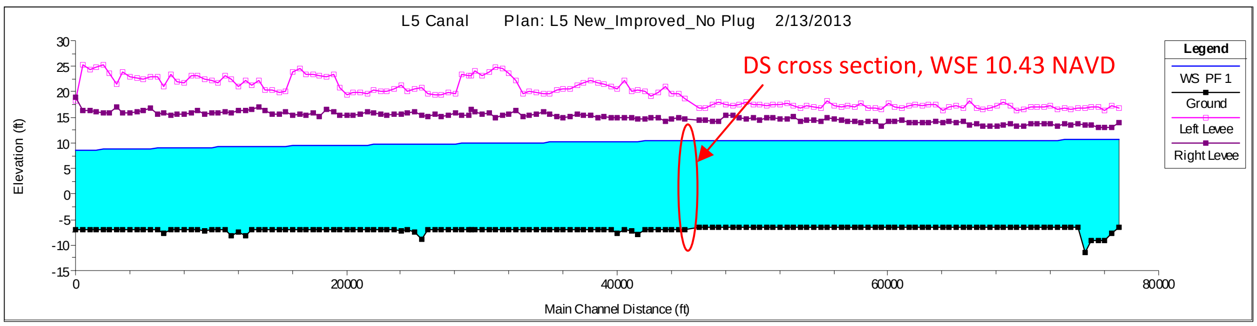


FIGURE A-41. FLOW PROFILE WITH SCENARIO 1 IMPROVEMENTS AND PLUG REMOVED (ELEVATIONS IN NAVD)

A.5.1.6 Conclusion

Modeling of the existing conditions of both the east and west portions of the L-5 canal indicated that neither portion can successfully convey the design flows given the assumed constraints. The maximum flow for the east and west are 350 cfs and 1,750 cfs, respectively. When modeled as a continuous reach assuming an inflow of 500 cfs from the L-6 canal, the western portion of L-5 can only convey 1,100 cfs (500 cfs from L-6 and 600 cfs from STA 3/4).

Scenario 1 included improvements to both the east and west portions of the L-5 canal to accommodate full L-6 and STA 3/4 conveyances, 500 cfs and 2,500 cfs (3,000 cfs total), respectively. The east portion of L-5 required an expansion to a bottom width of 50 ft and a deepening to bottom elevation of -5.1 ft NGVD (-6.5 ft NAVD). The west portion required an expansion to a bottom width of 100 ft and a deepening to bottom elevation -5.6 ft NGVD (-7.0 ft NAVD). The total excavation required for those improvements is 1,974,483 cubic yards. The model results from the improvements met all outlined assumptions and constraints.

Scenario 2 included improvements to only the west portion of the L-5 canal to accommodate full L-6 and STA 3/4 conveyances. Due to the lack of improvements east of the culverts, the 2,500 cfs lateral inflow from STA 3/4 west of the culverts into the L-5 canal creates a greater backwater effect than Scenario 1. The west portion of the canal must have a greater storage area to accommodate the increased flow in order to maintain a water surface elevation at or less than 12.0 ft NGVD on the most upstream (east) end of L-5. The improvements included a bottom width expansion to 100 ft and a deepening to -9.6 ft NGVD (-11 ft NAVD). The model results from the improvements met all outlined assumptions and constraints. The total excavation required for the improvement is 2,298,112 cubic yards, which exceeds the quantities for Scenario 1. Scenario 2 does not warrant further consideration.

Scenario 3 analyzed the west canal only, with no flow coming from the east (North New River Canal or L-6 canal). This scenario assumed the spillway gates were closed, or the existing plug remained. The flow analyzed was 3,000 cfs coming only from STA 3/4. The improvements included a bottom width expansion to 100 ft and a deepening to -5.6 ft NGVD (-7.0 ft NAVD). This improvement to the west differs from the improvements made in Scenario 1 in that it does not extend the entire length from Miami Canal to the culverts/plug. Improvements are made to cross sections beginning at the Miami Canal through RS 40193.85 (Figure A-38). The total excavation required for the improvement is 1,136,868 cubic yards. The model results from the improvements met all outlined assumptions and constraints.

Scenario 4 analyzed the canal with the assumption a pump station was constructed for the plug replacement in lieu of a gravity structure (spillway). This scenario improves both the east and west portions of the canal to convey their respective capacities of 500 cfs and 3,000 cfs. Each portion is improved independently of each other, since they are not hydraulically connected with a pump station in place. The eastern portion of the canal required an expansion to a bottom width of 50 ft and a deepening to bottom elevation of 0.4 ft NGVD (-1.0 ft NAVD). The west portion improvements were the same as those identified in Scenario 3. The total excavation required for those improvements is 1,370,578 cubic yards. The model results from the improvements met all outlined assumptions and constraints.

After all modeling was completed, it was determined that freeboard constraints were not the limiting constraint. The levee modifications to the north L-5 levee that were modeled were not further considered for cost analysis. As such, it is not recommended to make improvements to the low lying areas on the north levee. Additionally, standard criteria for vegetation-free zone on canal banks is 15 ft minimum, from land side to water side levee toes. Multiple cross sections throughout all options violate this criteria. Further refinement of canal alignment and template configuration will be conducted during detailed design.

All four scenarios produce relatively large quantities of excavation to convey all flows from the L-6 canal and STA 3/4. Options to consider and further develop costs:

1. Make improvements to both reaches of the L-5 canal and utilize a gravity structure at the existing plug location (Scenario 1).
2. No L-6 flows (Scenario 3). This option limits flows to only those from STA 3/4, which would not require canal improvements, degrading of the existing plug, construction of a control structure, and would only require minimal levee improvements to select locations.
3. Make improvements to both reaches of the L-5 canal and utilize a pump station at the existing plug location (Scenario 4). This would require fewer improvements to the eastern portion of L-5 as compared to Option 1, and would decrease the amount of total excavation.

Features	Option 1	Option 2	Option 3
L-6 Conveyance	X		X
L-5 plug remains		X	
L-5 gravity structure	X		
L-5 Pump Station			X
West L-5 Improvements	X	X	X
East L-5 Improvements	X		X
Excavation Quantities	1.97M cy	1.14M cy	1.37M cy

TABLE A-8. L-5 CANAL SUMMARY OF OPTIONS

		A.5.2 S-621 Spillway								
		STA 3/4 Outflow Canal								
Structure Design Criteria										
1	**Design Discharge =**	2500	cfs				**SPF Information**			
2	**Design Headwater Elev Hd =**	12.20	ft NGVD	Hd =	11.200	feet		SPF Discharge =		cfs
3	**Analize Headwater Elev He =**	12.20	ft NGVD	He =	11.22	ft		SPF Headwater =		ft NGVD
4	**Design Tailwater Elev =**	12.00	ft NGVD	He/H'd =	1.002			SPF Tailwater =		ft NGVD
5	**Crest Elevation =**	1.00	ft NGVD	h or Hs =	11	ft				
6	**Single Gate Crest Width =**	23	ft	Hs/He =	0.980		**Optimum Water Surface Elevations**			
7	**Number of gates =**	3		Delta H or hd =	0.2	feet		High headwater =		ft NGVD
	Net Crest Width =	69	feet	Delta-H /He=	0.01782			High Tailwater =		ft NGVD
8	**Intermediate Pier Width =**	3.25	feet					Low Headwater =		ft NGVD
9	**Upstr Canal Bottom Width =**	66.00	feet					Low Tailwater =		ft NGVD
10	**Upstr Bottom of channel Elev =**	-7.0	ft NGVD					Max Headwater =		ft NGVD
11	**Side Slope = 1 on**	2.0	ft NGVD					Lowest tailwater =		ft NGVD
12	**Natural Grade Upstream =**		ft NGVD							
13	**Natural Grade Downstream =**		ft NGVD				**Protection Elevations**			
14	**Highest Headwater el**	12.60	ft NGVD					Wave Surge at SPF =		feet
15	**Gate clearance above water**	1.00	feet					Structure Protection Elev =		ft NGVD
	Upstream Approach Velocity =	1.25	fps					Upstream Riprip Elev =		ft NGVD
								Downstream Riprip Elev =		ft NGVD
Crest Length Reduction due to Contractions			Warning!! Check for wave runup.					BreastWall Elevation =		ft NGVD
	From Plate 7 EM 1110-2-1603		Computed Ka and Kp from charts					Clearance Elevation =		ft NGVD
16	Pier Type (1, 2, 3 or 4)	2	Kp=	0.013						
	Number of Gates =	3	Ka=	0.174						
	Number of Piers =	2								
	Width of Gates =	23	feet							
	Height of Gates	12.6	feet	Recommended height						
17	Height of Gates	12.5	feet	***Designer's choice***			Area (h*L) =	759	sqft	
	Gate Aspect Ratio (about 2.0) =	1.84	OK				Unit Q q =	36.232	cfs/ft	
	Top of Gate elev	13.60	ft NGVD	Clearance Elev			Upstr Depth	19.2	feet	
	L=L'-2(N *Kp+Ka)He	64.51	feet				Upstr Avg Area	2004.48	sq ft	
	Crest discharge/foot q=	38.756	cfs/ft							
	Apron Width =	75.50	feet	Net Crest Width + Pier width(s)			OR Levee Elevation			
COEFFICIENT OF DISCHARGE COMPUTATION										
18	**Trial Upstr Apron Elev =**	-5	ft NGVD							
	Computed Free Discharge Coefficients		High or Low Ogee Weir?							
	Approach Apron Height P=	6.00	feet	Ratio P/Hd =	0.536	Apron Elevation Ok				
	Approach velocity =	1.93	fps							
	Coefficient of Free Discharge Cf =	3.879	<<<<<	**From Plate 31 EM 1110-2-1603**						
19	**Designer Discharge Coeff=**	3.7500	***Designers Judgement***							
	Free Discharge Qf= C*L*He^1.5	9,096	cfs - HDC 111-4/1; Is Hs/Hc < 0.4?			**->NO!, Must Use Submerged Discharge Qs**				
DISCHARGE REDUCTION FOR SUBMERGENCE FORMULAS OUTPUT										
Low Ogee Crests Discharge Coeficient Reduction: Submerged Flow										
	From Plate 33 EM 1110-2-1603									
20	**Trial Downstr Apron Elev =**	-5.00	ft NGVD							
	Corps Reduction Factor Data d=	17.00	feet	(Hd+d)/He =	1.53	H/He =	0.02			
21	**Corps % Reduction =**	71.96%	**<<<<Look up on EM 1110-2-1603 Plate 3-5 or HDC 111-4**							
				Coefficient						
	Corps Reduced Coefficient =	0.2804	x Cf =	1.088						
	USGS Reduced Coefficient Cs/C=	0.2796	x Cf =	1.048						
	SCS Reduced Coefficient qs/q =	0.1498								
DISCHARGE REDUCED FOR SUBMERGENCE										
		REQUIRED Discharge =	2,500	cfs (From original input)						
	Corps Qs = C x (% Reduction) x L x He^1.5 =		2,550	cfs *Warning! Assumed Apron Elev, Recheck after downstream Apron design.						
				Be sure to check Apron design and Re-enter Dwnstr Elev if Reqd.						
	USDWC Qs =Cs*L*He^1.5		2,543	cfs						
	SCS Qs = (qs/qf)*Qf =		1,362	cfs						
	Average Discharge		2,152	cfs						
	SFWMD Qs =Qf*(1-(Hs/He)^1.5)^0.385 =		2,369	cfs						
									k	
	D'Aubusons Qs= k*A*(2g(Hw-Tw)+V^2)^0.5 =		2,451	cfs	k from M 1110-2-1605 pg 5-14				0.85	
	Delta H/He< 0.2 Ok to use D'Aubusons Q		2,307	cfs	If Del H >1.0 then k = 0.85; if Del H < 1.0 K= 0				0.8	

	Apron Design	**Alternate**	**Ogee**							
		Controlling	**Design**							
		1	2							
22	Design Discharge	2500	2500	cfs	**Designers Choice; Choose higher computed discharge**					
23	Headwater for Apron =	12.2	12.20	feet,ngvd						
24	Tailwater for Apron =	12	12	feet,ngvd	**!!!Lowest tailwater with maximum discharge**					
	Trial Apron Elevation =	-5	-5	feet,ngvd	**Chosen at beginning of design process**					
25	Design Apron Elevation =	-5	-5	feet,ngvd	***Designers choice to Change Check line10***					
	Congugate Depth "E" =	17.20	17.20	feet						
	$q/(E^{1.5})$ =	0.543	0.543							
	D_2/E =	0.4773	0.4773		**Computed From Congugate Depth Curve**					
	Computed D2 =	8.21	8.21	feet	(1)					
	Actual D2 (Tw El - Apron El) =	17	17	feet	(2)					
	Designers Choice D2 =	17	17	feet						
	Alternate Design 1 d/D2 =	207.06%		**OK - Controlling D2 ratio > 85%**						
	Alternate Design 2 d/D2 =		207.06%	**Ok-Controlling D2 Ratio > 85%**						
	D1/E =	0.0711	0.0711		**Computed From Congugate Depth Curve**					
	Computed D1 =	1.222	1.222	feet						
	Velocity at D1 Depth =	31.708	31.708	fps						
	Frude no. F1 =	5.054	5.054			**Jump Classified as =**	**Steady**	**jump**		
	Design Apron Elevation =	-5	-5	feet						
	Designed Apron Width =	75.50	75.50	feet						
	Average Apron Velocity =	1.95	1.95	fps						
	Hydraulic Jump Length No Baffles =	49.42	49.42	feet	**On Flat Floor No Baffles or Endsill EM1110-2-1603 (7-1)**					
	Length of basin with Baffles =	19.44	19.44	feet	**Lb=K*D1*F1^1.5 K=1.4 EM1110-2-1603**					
	Apron Length with Baffles =	27.78	27.78	feet	**Lb=K*D1*F1^1.5 K=2.0 EM1110-2-1603**					
	Apron Length (2.5xD2) =	42.50	42.50	feet	**Previous Recommendations**					
	Apron Length (3xD2) =	51.00	51.00	feet	**Previous Recommendations**					
25	Designer Apron Length =	51.00	25.00	feet	***Designers choice***		**Minimum Design OK**			
	Baffle Block Design									
	Baffle Block Height =	2.83	1.30	feet	**EM-1110-2-1603 Plate 7-4**					
26	Designer Block Height D1 =	2.50	1.50	feet	***Designers choice***					
27	Design Width of Block (D1) =	2.00	2.00	feet	***Designers choice***			37.75		
	Top of Baffle Elevation =	-2.50	-3.50	feet,ngvd						
	Distance from Toe of Ogee to Upstream Face of First Row of Blocks									
	Distance from Toe of Ogee =	26.02	12.89	feet	EM-1110-2-1603 Plate 7-4					
	Distance from Toe of Ogee =	12.31	12.31	feet	First Row of Blocks from OgeeWeir 1.5*D2					
28	Designer Distance to 1st row =	25.00	25.00	feet	***Designers choice***					
	Distance from Toe of Ogee to Upstream face of Second Row of Blocks									
	Distance from Ogee toe=	20.81	20.81	feet	Second Row of Blocks from OgeeWeir First Row + 0.5*D2					
29	Designer Distance to 2nd Row =	20.00	20.00	feet	***Designers choice***					
End Sill Design				Edsill width =	1.00	feet				
	End Sill Height (D2/12) =	1.417	1.417							
	End Sill Height (D1/2) =	0.611	0.611			Slope =	10.784%			
	Recommended Height =	0.611	0.611							
30	Designer Sill Height =	0.50	0.50	feet	***Designers Choice***					
	End Sill Elevation	-4.5	-4.5	feet,ngvd						
	Additional Length ot basin	2.00	2.00	1 on 1 slope from apron floor to top of endsill						
	Total Apron Length from Ogee Toe =	53.00	27.00	feet						
	End Sill Froude No. =	0.09	0.09		ok					
	End Sill Velocity =	2.01	2.01	fps		Velocity < 9.0 fps, Design OK				
31	RipRap Velocity	5.00	5.00	fps	***Designers Recommendation***					

		A.5.2 S-622 Spillway								
		L-5 Canal Plug Replacement								
Structure Design Criteria										
1	**Design Discharge =**	500	cfs				**SPF Information**			
2	**Design Headwater Elev Hd =**	11.93	ft NGVD	Hd =	6.930	feet		SPF Discharge =		cfs
3	**Analize Headwater Elev He =**	11.93	ft NGVD	He =	6.93	ft		SPF Headwater =		ft NGVD
4	**Design Tailwater Elev =**	11.83	ft NGVD	He/H'd =	1.000			SPF Tailwater =		ft NGVD
5	**Crest Elevation =**	5.00	ft NGVD	h or Hs =	6.83	ft				
6	**Single Gate Crest Width =**	15	ft	Hs/He =	0.985		**Optimum Water Surface Elevations**			
7	**Number of gates =**	3		Delta H or hd =	0.1	feet		High headwater =		ft NGVD
	Net Crest Width =	45	feet	Delta-H /He=	0.01443			High Tailwater =		ft NGVD
8	**Intermediate Pier Width =**	3.25	feet					Low Headwater =		ft NGVD
9	**Upstr Canal Bottom Width =**	75.00	feet					Low Tailwater =		ft NGVD
10	**Upstr Bottom of channel Elev =**	-5.1	ft NGVD					Max Headwater =		ft NGVD
11	**Side Slope = 1 on**	2.0	ft NGVD					Lowest tailwater =		ft NGVD
12	**Natural Grade Upstream =**	6	ft NGVD							
13	**Natural Grade Downstream =**	6	ft NGVD				**Protection Elevations**			
14	**Highest Headwater el**	12.00	ft NGVD					Wave Surge at SPF =		feet
15	**Gate clearance above water**	1.00	feet					Structure Protection Elev =		ft NGVD
	Upstream Approach Velocity =	0.27	fps					Upstream Riprip Elev =		ft NGVD
								Downstream Riprip Elev =		ft NGVD
Crest Length Reduction due to Contractions			Warning!! Check for wave runup.					BreastWall Elevation =		ft NGVD
	From Plate 7 EM 1110-2-1603		Computed Ka and Kp from charts					Clearance Elevation =		ft NGVD
16	Pier Type (1, 2, 3 or 4)	2	Kp=	0.013						
	Number of Gates =	3	Ka=	0.174						
	Number of Piers =	2								
	Width of Gates =	15	feet							
	Height of Gates	8	feet	Recommended height						
17	Height of Gates	10	feet	***Designer's choice***			Area (h*L) =	307.35	sqft	
	Gate Aspect Ratio (about 2.0) =	1.50	OK				Unit Q q =	11.111	cfs/ft	
	Top of Gate elev	13.00	ft NGVD	Clearance Elev			Upstr Depth	17.03	feet	
	L=L'-2(N *Kp+Ka)He	42.22	feet				Upstr Avg Area	1857.29	sq ft	
	Crest discharge/foot q=	11.842	cfs/ft							
	Apron Width =	51.50	feet	Net Crest Width + Pier width(s)			OR Levee Elevation			
COEFFICIENT OF DISCHARGE COMPUTATION										
18	**Trial Upstr Apron Elev =**	0	ft NGVD							
	Computed Free Discharge Coefficients		High or Low Ogee Weir?							
	Approach Apron Height P=	5.00	feet	Ratio P/Hd =	0.722	Apron Elevation Ok				
	Approach velocity =	0.81	fps							
	Coefficient of Free Discharge Cf =	3.903	<<<<<	**From Plate 31 EM 1110-2-1603**						
19	**Designer Discharge Coeff=**	3.7500	***Designers Judgement***							
	Free Discharge Qf= C*L*He^1.5	2,889	cfs - HDC 111-4/1; Is Hs/Hc < 0.4?			**->NO!, Must Use Submerged Discharge Qs**				
DISCHARGE REDUCTION FOR SUBMERGENCE FORMULAS OUTPUT										
Low Ogee Crests Discharge Coeficient Reduction: Submerged Flow										
	From Plate 33 EM 1110-2-1603									
20	**Trial Downstr Apron Elev =**	0.00	ft NGVD							
	Corps Reduction Factor Data d=	11.83	feet	(Hd+d)/He =	1.72	H/He =	0.01			
21	**Corps % Reduction =**	77.67%	**<<<<Look up on EM 1110-2-1603 Plate 3-5 or HDC 111-4**							
				Coefficient						
	Corps Reduced Coefficient =	0.2233	x Cf =	0.872						
	USGS Reduced Coefficient Cs/C=	0.2043	x Cf =	0.766						
	SCS Reduced Coefficient qs/q =	0.1094								
DISCHARGE REDUCED FOR SUBMERGENCE										
		REQUIRED Discharge =	500	cfs (From original input)						
	Corps Qs = C x (% Reduction) x L x He^1.5 =		645	cfs *Warning! Assumed Apron Elev, Recheck after downstream Apron design.						
				Be sure to check Apron design and Re-enter Dwnstr Elev if Reqd.						
	USDWC Qs =Cs*L*He^1.5		590	cfs						
	SCS Qs = (qs/qf)*Qf =		316	cfs						
	Average Discharge		517	cfs						
	SFWMD Qs =Qf*(1-(Hs/He)^1.5)^0.385 =		667	cfs						
									k	
	D'Aubusons Qs= k*A*(2g(Hw-Tw)+V^2)^0.5 =		667	cfs	k from M 1110-2-1605 pg 5-14				0.85	
	Delta H/He< 0.2 Ok to use D'Aubusons Q		627	cfs	If Del H >1.0 then k = 0.85; if Del H < 1.0 K= 0				0.8	

	Apron Design	**Alternate**	**Ogee**							
		Controlling	**Design**							
		1	2							
22	Design Discharge	500	1000	cfs	**Designers Choice; Choose higher computed discharge**					
23	Headwater for Apron =	11.93	11.93	feet,ngvd						
24	Tailwater for Apron =	11.93	11.83	feet,ngvd	**!!!Lowest tailwater with maximum discharge**					
	Trial Apron Elevation =	0	0	feet,ngvd	**Chosen at beginning of design process**					
25	Design Apron Elevation =	0	0	feet,ngvd	***Designers choice to Change Check line10***					
	Congugate Depth "E" =	11.93	11.93	feet						
	$q/(E^{1.5})$ =	0.287	0.575							
	D_2/E =	0.3549	0.4894		**Computed From Congugate Depth Curve**					
	Computed D2 =	4.23	5.84	feet	(1)					
	Actual D2 (Tw El - Apron El) =	11.93	11.83	feet	(2)					
	Designers Choice D2 =	11	11	feet						
	Alternate Design 1 d/D2 =	259.78%		**OK - Controlling D2 ratio > 85%**						
	Alternate Design 2 d/D2 =		202.61%	**Ok-Controlling D2 Ratio > 85%**						
	D1/E =	0.0355	0.0755		**Computed From Congugate Depth Curve**					
	Computed D1 =	0.423	0.900	feet						
	Velocity at D1 Depth =	27.972	26.306	fps						
	Frude no. F1 =	7.576	4.886		**Jump Classified as =**		**Steady**	**jump**		
	Design Apron Elevation =	0	0	feet						
	Designed Apron Width =	51.50	51.50	feet						
	Average Apron Velocity =	0.81	1.64	fps						
	Hydraulic Jump Length No Baffles =	25.66	34.03	feet	**On Flat Floor No Baffles or Endsill EM1110-2-1603 (7-1)**					
	Length of basin with Baffles =	12.36	13.61	feet	**Lb=K*D1*F1^1.5**	**K=1.4**	**EM1110-2-1603**			
	Apron Length with Baffles =	17.66	19.45	feet	**Lb=K*D1*F1^1.5**	**K=2.0**	**EM1110-2-1603**			
	Apron Length (2.5xD2) =	27.50	27.50	feet	**Previous Recommendations**					
	Apron Length (3xD2) =	33.00	33.00	feet	**Previous Recommendations**					
25	Designer Apron Length =	33.00	33.00	feet	***Designers choice***		**Minimum Design OK**			
	Baffle Block Design									
	Baffle Block Height =	1.83	0.97	feet	**EM-1110-2-1603 Plate 7-4**					
26	Designer Block Height D1 =	2.00	1.00	feet	***Designers choice***					
27	Design Width of Block (D1) =	2.00	2.00	feet	***Designers choice***			25.75		
	Top of Baffle Elevation =	2.00	1.00	feet,ngvd						
	Distance from Toe of Ogee to Upstream Face of First Row of Blocks									
	Distance from Toe of Ogee =	19.61	8.79	feet	EM-1110-2-1603 Plate 7-4					
	Distance from Toe of Ogee =	6.35	8.76	feet	First Row of Blocks from OgeeWeir 1.5*D2					
28	Designer Distance to 1st row =	20.00	9.00	feet	***Designers choice***					
	Distance from Toe of Ogee to Upstream face of Second Row of Blocks									
	Distance from Ogee toe=	11.85	14.26	feet	Second Row of Blocks from OgeeWeir First Row + 0.5*D2					
29	Designer Distance to 2nd Row =	12.00	14.50	feet	***Designers choice***					
End Sill Design				Edsill width =	1.00	feet				
	End Sill Height (D2/12) =	0.994	0.986							
	End Sill Height (D1/2) =	0.212	0.450			Slope =	13.636%			
	Recommended Height =	0.212	0.450							
30	Designer Sill Height =	0.50	0.50	feet	***Designers Choice***					
	End Sill Elevation	0.5	0.5	feet,ngvd						
	Additional Length ot basin	2.00	2.00	1 on 1 slope from apron floor to top of endsill						
	Total Apron Length from Ogee Toe =	35.00	35.00	feet						
	End Sill Froude No. =	0.04	0.09		ok					
	End Sill Velocity =	0.85	1.71	fps		Velocity < 9.0 fps, Design OK				
31	RipRap Velocity	5.00	5.00	fps	***Designers Recommendation***					

A.6 BLUE/GREE/YELLOW LINES

A.6.1 S-333N Spillway
L-67/L-29 Canal

Structure Design Criteria										
1	**Design Discharge =**	1150	cfs				**SPF Information**			
2	**Design Headwater Elev Hd =**	7.50	ft NGVD	Hd =	10.600	feet		SPF Discharge =		cfs
3	**Analize Headwater Elev He =**	7.50	ft NGVD	He =	10.61	ft		SPF Headwater =		ft NGVD
4	**Design Tailwater Elev =**	7.00	ft NGVD	He/H'd =	1.001			SPF Tailwater =		ft NGVD
5	**Crest Elevation =**	-3.10	ft NGVD	h or Hs =	10.1	ft				
6	**Single Gate Crest Width =**	29	ft	Hs/He =	0.952		**Optimum Water Surface Elevations**			
7	**Number of gates =**	1		Delta H or hd =	0.5	feet		High headwater =		ft NGVD
	Net Crest Width =	29	feet	Delta-H /He=	0.04713			High Tailwater =		ft NGVD
8	**Intermediate Pier Width =**	3.25	feet					Low Headwater =		ft NGVD
9	**Upstr Canal Bottom Width =**	35.00	feet					Low Tailwater =		ft NGVD
10	**Upstr Bottom of channel Elev =**	-10.0	ft NGVD					Max Headwater =		ft NGVD
11	**Side Slope = 1 on**	3.0	ft NGVD					Lowest tailwater =		ft NGVD
12	**Natural Grade Upstream =**	15	ft NGVD							
13	**Natural Grade Downstream =**	15	ft NGVD				**Protection Elevations**			
14	**Highest Headwater el**	7.50	ft NGVD					Wave Surge at SPF =		feet
15	**Gate clearance above water**	1.00	feet					Structure Protection Elev =		ft NGVD
	Upstream Approach Velocity =	0.75	fps					Upstream Riprip Elev =		ft NGVD
								Downstream Riprip Elev =		ft NGVD
Crest Length Reduction due to Contractions			Warning!! Check for wave runup.					BreastWall Elevation =		ft NGVD
	From Plate 7 EM 1110-2-1603		Computed Ka and Kp from charts					Clearance Elevation =		ft NGVD
16	Pier Type (1, 2, 3 or 4)	2	Kp=	0.013						
	Number of Gates =	1	Ka=	0.174						
	Number of Piers =	0								
	Width of Gates =	29	feet							
	Height of Gates	11.6	feet	Recommended height						
17	Height of Gates	14.6	feet	***Designer's choice***			Area (h*L) =	292.9	sqft	
	Gate Aspect Ratio (about 2.0) =	1.99	OK				Unit Q q =	39.655	cfs/ft	
	Top of Gate elev	8.50	ft NGVD	Clearance Elev			Upstr Depth	17.5	feet	
	L=L'-2(N *Kp+Ka)He	25.31	feet				Upstr Avg Area	1531.25	sq ft	
	Crest discharge/foot q=	45.443	cfs/ft							
	Apron Width =	29.00	feet	Net Crest Width + Pier width(s)			OR Levee Elevation			
COEFFICIENT OF DISCHARGE COMPUTATION										
18	**Trial Upstr Apron Elev =**	-6	ft NGVD							
	Computed Free Discharge Coefficients		High or Low Ogee Weir?							
	Approach Apron Height P=	2.90	feet	Ratio P/Hd =	0.274	P is too low -Lower Approach Apron				
	Approach velocity =	2.94	fps							
	Coefficient of Free Discharge Cf =	3.843	<<<<<	**From Plate 31 EM 1110-2-1603**						
19	**Designer Discharge Coeff=**	3.7500	***Designers Judgement***							
	Free Discharge Qf= C*L*He^1.5	3,279	cfs - HDC 111-4/1; Is Hs/Hc < 0.4?			**->NO!, Must Use Submerged Discharge Qs**				
DISCHARGE REDUCTION FOR SUBMERGENCE FORMULAS OUTPUT										
Low Ogee Crests Discharge Coeficient Reduction: Submerged Flow										
	From Plate 33 EM 1110-2-1603									
20	**Trial Downstr Apron Elev =**	-6.00	ft NGVD							
	Corps Reduction Factor Data d=	13.00	feet	(Hd+d)/He =	1.27	H/He =	0.05			
21	**Corps % Reduction =**	47.71%	**<<<<Look up on EM 1110-2-1603 Plate 3-5 or HDC 111-4**							
				Coefficient						
	Corps Reduced Coefficient =	0.5229	x Cf =	2.010						
	USGS Reduced Coefficient Cs/C=	0.4458	x Cf =	1.672						
	SCS Reduced Coefficient qs/q =	0.3217								
DISCHARGE REDUCED FOR SUBMERGENCE										
		REQUIRED Discharge =	1,150	cfs (From original input)						
	Corps Qs = C x (% Reduction) x L x He^1.5 =		1,715	cfs *Warning! Assumed Apron Elev, Recheck after downstream Apron design.						
				Be sure to check Apron design and Re-enter Dwnstr Elev if Reqd.						
	USDWC Qs =Cs*L*He^1.5		1,462	cfs						
	SCS Qs = (qs/qf)*Qf =		1,055	cfs						
	Average Discharge		1,411	cfs						
	SFWMD Qs =Qf*(1-(Hs/He)^1.5)^0.385 =		1,191	cfs						
									k	
	D'Aubusons Qs= k*A*(2g(Hw-Tw)+V^2)^0.5 =		1,425	cfs	k from M 1110-2-1605 pg 5-14				0.85	
	Delta H/He< 0.2 Ok to use D'Aubusons Q		1,341	cfs	If Del H >1.0 then k = 0.85; if Del H < 1.0 K= 0				0.8	

	Apron Design	**Alternate**	**Ogee**							
		Controlling	**Design**							
		1	2							
22	Design Discharge	1150	1650	cfs	**Designers Choice; Choose higher computed discharge**					
23	Headwater for Apron =	7.5	7.50	feet,ngvd						
24	Tailwater for Apron =	7	7	feet,ngvd	**!!!Lowest tailwater with maximum discharge**					
	Trial Apron Elevation =	-6	-6	feet,ngvd	**Chosen at beginning of design process**					
25	Design Apron Elevation =	-6	-6	feet,ngvd	***Designers choice to Change Check line10***					
	Congugate Depth "E" =	13.50	13.50	feet						
	$q/(E^{1.5})$ =	0.916	1.314							
	D_2/E =	0.5962	0.6839		**Computed From Congugate Depth Curve**					
	Computed D2 =	8.05	9.23	feet (1)						
	Actual D2 (Tw El - Apron El) =	13	13	feet (2)						
	Designers Choice D2 =	13	13	feet						
	Alternate Design 1 d/D2 =	161.52%		**OK - Controlling D2 ratio > 85%**						
	Alternate Design 2 d/D2 =		140.81%	**Ok-Controlling D2 Ratio > 85%**						
	D1/E =	0.1239	0.1803		**Computed From Congugate Depth Curve**					
	Computed D1 =	1.673	2.434	feet						
	Velocity at D1 Depth =	27.160	26.785	fps						
	Frude no. F1 =	3.700	3.025			**Jump Classified as =**	**Oscillating**	**jump**		
	Design Apron Elevation =	-6	-6	feet						
	Designed Apron Width =	29.00	29.00	feet						
	Average Apron Velocity =	3.05	4.38	fps						
	Hydraulic Jump Length No Baffles =	41.68	44.83	feet	**On Flat Floor No Baffles or Endsill EM1110-2-1603 (7-1)**					
	Length of basin with Baffles =	16.67	17.93	feet	**Lb=K*D1*F1^1.5 K=1.4**		**EM1110-2-1603**			
	Apron Length with Baffles =	23.82	25.62	feet	**Lb=K*D1*F1^1.5 K=2.0**		**EM1110-2-1603**			
	Apron Length (2.5xD2) =	32.50	32.50	feet	**Previous Recommendations**					
	Apron Length (3xD2) =	39.00	39.00	feet	**Previous Recommendations**					
25	Designer Apron Length =	39.00	39.00	feet	***Designers choice***		**Minimum Design OK**			
	Baffle Block Design									
	Baffle Block Height =	2.17	1.54	feet	**EM-1110-2-1603 Plate 7-4**					
26	Designer Block Height D1 =	1.00	2.00	feet	***Designers choice***					
27	Design Width of Block (D1) =	2.00	2.00	feet	***Designers choice***			14.50		
	Top of Baffle Elevation =	-5.00	-4.00	feet,ngvd						
	Distance from Toe of Ogee to Upstream Face of First Row of Blocks									
	Distance from Toe of Ogee =	19.50	13.85	feet	EM-1110-2-1603 Plate 7-4					
	Distance from Toe of Ogee =	12.07	13.85	feet	First Row of Blocks from OgeeWeir 1.5*D2					
28	Designer Distance to 1st row =	19.50	14.00	feet	***Designers choice***					
	Distance from Toe of Ogee to Upstream face of Second Row of Blocks									
	Distance from Ogee toe=	18.57	20.35	feet	Second Row of Blocks from OgeeWeir First Row + 0.5*D2					
29	Designer Distance to 2nd Row =	19.00	20.50	feet	***Designers choice***					
End Sill Design				Edsill width =	1.00	feet				
	End Sill Height (D2/12) =	1.083	1.083							
	End Sill Height (D1/2) =	0.837	1.217			Slope =	6.154%			
	Recommended Height =	0.837	1.083							
30	Designer Sill Height =	0.50	1.00	feet	***Designers Choice***					
	End Sill Elevation	-5.5	-5	feet,ngvd						
	Additional Length ot basin	2.00	3.00	1 on 1 slope from apron floor to top of endsill						
	Total Apron Length from Ogee Toe =	41.00	42.00	feet						
	End Sill Froude No. =	0.16	0.24		ok					
	End Sill Velocity =	3.17	4.74	fps		Velocity < 9.0 fps, Design OK				
31	RipRap Velocity	5.00	5.00	fps	***Designers Recommendation***					

		A.6.2 S-355W Spillway								
		L-29 Canal								
Structure Design Criteria										
1	**Design Discharge =**	1230	cfs				**SPF Information**			
2	**Design Headwater Elev Hd =**	9.70	ft NGVD	Hd =	5.700	feet	SPF Discharge =			cfs
3	**Analize Headwater Elev He =**	9.70	ft NGVD	He =	5.71	ft	SPF Headwater =			ft NGVD
4	**Design Tailwater Elev =**	8.70	ft NGVD	He/H'd =	1.001		SPF Tailwater =			ft NGVD
5	**Crest Elevation =**	4.00	ft NGVD	h or Hs =	4.7	ft				
6	**Single Gate Crest Width =**	12	ft	Hs/He =	0.823		**Optimum Water Surface Elevations**			
7	**Number of gates =**	3		Delta H or hd =	1	feet	High headwater =			ft NGVD
	Net Crest Width =	36	feet	Delta-H /He=	0.17521		High Tailwater =			ft NGVD
8	**Intermediate Pier Width =**	3.25	feet				Low Headwater =			ft NGVD
9	**Upstr Canal Bottom Width =**	50.00	feet				Low Tailwater =			ft NGVD
10	**Upstr Bottom of channel Elev =**	-7.6	ft NGVD				Max Headwater =			ft NGVD
11	**Side Slope = 1 on**	3.0	ft NGVD				Lowest tailwater =			ft NGVD
12	**Natural Grade Upstream =**		ft NGVD							
13	**Natural Grade Downstream =**		ft NGVD				**Protection Elevations**			
14	**Highest Headwater el**	9.70	ft NGVD				Wave Surge at SPF =			feet
15	**Gate clearance above water**	1.00	feet				Structure Protection Elev =			ft NGVD
	Upstream Approach Velocity =	0.70	fps				Upstream Riprip Elev =			ft NGVD
							Downstream Riprip Elev =			ft NGVD
Crest Length Reduction due to Contractions			Warning!! Check for wave runup.				BreastWall Elevation =			ft NGVD
	From Plate 7 EM 1110-2-1603		Computed Ka and Kp from charts				Clearance Elevation =			ft NGVD
16	Pier Type (1, 2, 3 or 4)	2	Kp=	0.013						
	Number of Gates =	3	Ka=	0.174						
	Number of Piers =	2								
	Width of Gates =	12	feet							
	Height of Gates	6.7	feet	Recommended height						
17	Height of Gates	8	feet	***Designer's choice***			Area (h*L) =	169.2	sqft	
	Gate Aspect Ratio (about 2.0) =	1.50	OK				Unit Q q =	34.167	cfs/ft	
	Top of Gate elev	10.70	ft NGVD	Clearance Elev			Upstr Depth	17.3	feet	
	L=L'-2(N *Kp+Ka)He	33.71	feet				Upstr Avg Area	1762.87	sq ft	
	Crest discharge/foot q=	36.484	cfs/ft							
	Apron Width =	42.50	feet	Net Crest Width + Pier width(s)			OR Levee Elevation			
COEFFICIENT OF DISCHARGE COMPUTATION										
18	**Trial Upstr Apron Elev =**	-4	ft NGVD							
	Computed Free Discharge Coefficients		High or Low Ogee Weir?							
	Approach Apron Height P=	8.00	feet	Ratio P/Hd =	1.404	Apron Elevation Ok				
	Approach velocity =	2.11	fps							
	Coefficient of Free Discharge Cf =	3.953	<<<<<	**From Plate 31 EM 1110-2-1603**						
19	**Designer Discharge Coeff=**	3.7500	***Designers Judgement***							
	Free Discharge Qf= C*L*He^1.5	1,724	cfs - HDC 111-4/1; Is Hs/Hc < 0.4?			**->NO!, Must Use Submerged Discharge Qs**				
DISCHARGE REDUCTION FOR SUBMERGENCE FORMULAS OUTPUT										
Low Ogee Crests Discharge Coeficient Reduction: Submerged Flow										
	From Plate 33 EM 1110-2-1603									
20	**Trial Downstr Apron Elev =**	-4.00	ft NGVD							
	Corps Reduction Factor Data d=	12.70	feet	(Hd+d)/He =	2.40	H/He =	0.18			
21	**Corps % Reduction =**	9.55%	**<<<<Look up on EM 1110-2-1603 Plate 3-5 or HDC 111-4**							
				Coefficient						
	Corps Reduced Coefficient =	0.9045	x Cf =	3.575						
	USGS Reduced Coefficient Cs/C=	0.7531	x Cf =	2.824						
	SCS Reduced Coefficient qs/q =	0.8122								
DISCHARGE REDUCED FOR SUBMERGENCE										
	REQUIRED Discharge =		1,230	cfs (From original input)						
	Corps Qs = C x (% Reduction) x L x He^1.5 =		1,559	cfs *Warning! Assumed Apron Elev, Recheck after downstream Apron design.						
				Be sure to check Apron design and Re-enter Dwnstr Elev if Reqd.						
	USDWC Qs =Cs*L*He^1.5		1,298	cfs						
	SCS Qs = (qs/qf)*Qf =		1,400	cfs						
	Average Discharge		1,419	cfs						
	SFWMD Qs =Qf*(1-(Hs/He)^1.5)^0.385 =		1,018	cfs						
									k	
	D'Aubusons Qs= k*A*(2g(Hw-Tw)+V^2)^0.5 =		1,159	cfs	k from M 1110-2-1605 pg 5-14				0.85	
	Delta H/He< 0.2 Ok to use D'Aubusons Q		1,159	cfs	If Del H >1.0 then k = 0.85; if Del H < 1.0 K= 0				0.85	

	Apron Design	Alternate Controlling	Ogee Design							
		1	2							
22	Design Discharge	1230	1500	cfs	Designers Choice; Choose higher computed discharge					
23	Headwater for Apron =	9.7	9.70	feet,ngvd						
24	Tailwater for Apron =	8.7	8.7	feet,ngvd	**!!!Lowest tailwater with maximum discharge**					
	Trial Apron Elevation =	-4	-4	feet,ngvd	**Chosen at beginning of design process**					
25	Design Apron Elevation =	-4	-4	feet,ngvd	***Designers choice to Change Check line10***					
	Congugate Depth "E" =	13.70	13.70	feet						
	$q/(E^{1.5})$ =	0.719	0.877							
	D_2/E =	0.5402	0.5861		**Computed From Congugate Depth Curve**					
	Computed D2 =	7.40	8.03	feet	(1)					
	Actual D2 (Tw El - Apron El) =	12.7	12.7	feet	(2)					
	Designers Choice D2 =	12	12	feet						
	Alternate Design 1 d/D2 =	162.14%		**OK - Controlling D2 ratio > 85%**						
	Alternate Design 2 d/D2 =		158.16%	**Ok-Controlling D2 Ratio > 85%**						
	D1/E =	0.0937	0.1184		**Computed From Congugate Depth Curve**					
	Computed D1 =	1.284	1.622	feet						
	Velocity at D1 Depth =	28.413	27.432	fps						
	Frude no. F1 =	4.419	3.796			**Jump Classified as =**	**Oscillating**	**jump**		
	Design Apron Elevation =	-4	-4	feet						
	Designed Apron Width =	42.50	42.50	feet						
	Average Apron Velocity =	2.28	2.78	fps						
	Hydraulic Jump Length No Baffles =	41.74	41.98	feet	**On Flat Floor No Baffles or Endsill EM1110-2-1603 (7-1)**					
	Length of basin with Baffles =	16.70	16.79	feet	**Lb=K*D1*F1^1.5 K=1.4 EM1110-2-1603**					
	Apron Length with Baffles =	23.85	23.99	feet	**Lb=K*D1*F1^1.5 K=2.0 EM1110-2-1603**					
	Apron Length (2.5xD2) =	30.00	30.00	feet	**Previous Recommendations**					
	Apron Length (3xD2) =	36.00	36.00	feet	**Previous Recommendations**					
25	Designer Apron Length =	36.00	36.00	feet	***Designers choice***		**Minimum Design OK**			
	Baffle Block Design									
	Baffle Block Height =	2.00	1.34	feet	**EM-1110-2-1603 Plate 7-4**					
26	Designer Block Height D1 =	2.00	1.50	feet	***Designers choice***					
27	Design Width of Block (D1) =	2.00	2.00	feet	***Designers choice***			21.25		
	Top of Baffle Elevation =	-2.00	-2.50	feet,ngvd						
	Distance from Toe of Ogee to Upstream Face of First Row of Blocks									
	Distance from Toe of Ogee =	18.00	12.04	feet	EM-1110-2-1603 Plate 7-4					
	Distance from Toe of Ogee =	11.10	12.04	feet	First Row of Blocks from OgeeWeir 1.5*D2					
28	Designer Distance to 1st row =	18.00	12.00	feet	***Designers choice***					
	Distance from Toe of Ogee to Upstream face of Second Row of Blocks									
	Distance from Ogee toe=	17.10	18.04	feet	Second Row of Blocks from OgeeWeir First Row + 0.5*D2					
29	Designer Distance to 2nd Row =	17.00	18.00	feet	***Designers choice***					
End Sill Design				Edsill width =	1.00	feet				
	End Sill Height (D2/12) =	1.058	1.058							
	End Sill Height (D1/2) =	0.642	0.811			Slope =	20.833%			
	Recommended Height =	0.642	0.811							
30	Designer Sill Height =	0.50	1.00	feet	***Designers Choice***					
	End Sill Elevation	-3.5	-3	feet,ngvd						
	Additional Length ot basin	2.00	3.00	1 on 1 slope from apron floor to top of endsill						
	Total Apron Length from Ogee Toe =	38.00	39.00	feet						
	End Sill Froude No. =	0.12	0.16		ok					
	End Sill Velocity =	2.37	3.02	fps		Velocity < 9.0 fps, Design OK				
31	RipRap Velocity	5.00	5.00	fps	***Designers Recommendation***					

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ANNEX A-2: HYDROLOGIC MODELING

USACE Jacksonville District,
Water Resources Engineering Branch
July 2013

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List of Additional Supporting References to Annex A-2:

Reference 1: CEPP Modeling Strategy (October 2012)

Reference 2: RSM-BN and RSM-GL Modeling Assumptions Tables

Reference 3: RSM-GL Reference Maps

Reference 4: RSM-BN Water Budget Maps for Baselines and Alt4R2

Reference 5: RSM-GL Water Budget Maps for Baselines and Alt4R2

Reference 6: RSM-GL Hydroperiod, Ponding, and Difference Maps for Baselines and Alt4R2

1. Hydrologic and Hydraulic Modeling Strategy Development

The primary application of models in the CEPP is for the assessment of regional-level hydrologic planning. More detailed models were also applied to address specific questions related to hydraulic and water quality constraints. The CEPP modeling tools were jointly selected by the USACE Jacksonville District (SAJ) and the South Florida Water Management District (SFWMD) in October-November 2011 based on their collective capability to provide adequate hydrologic information to conduct evaluations of the entire south Florida system for the needs of the CEPP. Due to the time required to complete prerequisite model documentation, documentation review, and compilation of this model validation review package, the expedited CEPP schedule did not afford the opportunity to submit the proposed modeling tools for USACE Engineering software validation evaluation prior to execution of the modeling strategy and application of the initial recommended modeling tool suite, which initiated in January 2012. However, prior to implementation of the CEPP modeling, the CEPP modeling strategy was vetted through USACE at the SAJ District, South Atlantic Division (SAD), and Headquarters (HQ) levels through the prior CEPP periodic in-progress reviews (IPR-1 in December 2010; IPR-2 in January 2012) and CEPP Decision Point 1 vertical coordination meeting (January 2012). Prior to completion of the hydrologic modeling of the CEPP final array of alternatives, all CEPP modeling tools were reviewed and approved for use through either the USACE Engineering software validation process or through the CEPP Agency Technical Review (ATR) process, as further documented in Section A.8.1.1 of the Engineering Appendix.

The CEPP modeling strategy centered around use of a decoupled link-node model Regional Simulation Model for Basins (RSM-BN) for the EAA, Stormwater Treatment Areas (STAs) and the northern estuaries, in combination with a detailed meshed Regional Simulation Model for the Glades and Lower East Coast Service Areas (RSM-GL) for the Water Conservation Areas (WCAs), Everglades National Park (ENP) and the Lower East Coast (refer to section A.8.1.2 of the Engineering Appendix for additional documentation of the RSM models). The CEPP modeling strategy provides an overview of the modeling tools, including maps of the model domains, applied throughout the plan formulation process and how the tools were applied in support of the CEPP planning process (refer to Reference 1 of this Hydrologic Modeling Annex). Preliminary screening assessments for Lake Okeechobee, the northern estuaries, and the impoundment storage within the Everglades Agricultural Area (EAA), collectively referred to as the "North of the Red Line components," utilized the Reservoir Sizing and Operations Screening (RESOPS) model, the Lake Okeechobee Operations Screening (LOOPS) model, and the C-43 Spreadsheet Model. Preliminary screening assessments for the Water Conservation Areas (WCAs) and Everglades National Park (ENP), collectively referred to as the "South of the Red Line components" (including the components at the EAA/WCA Red Line boundary, in addition to the Green/Blue/Yellow Line components) utilized the iModel tool and limited-scope sensitivity simulations using the RSM-GL. For the final array of alternatives, analysis of the North of Red Line components and the South of the Red Line components were conducted using the RSM-BN and the RSM-GL, respectively. The RSM-GL model was also used for performance evaluation within the Lower East Coast Service Areas, areas which were not encompassed within the domain of the iModel. The complete RSM-GL model calibration and verification report, which was completed in December 2011 as part of the Decomp PIR 1 project modeling effort, is posted with the Decomp project documentation report (Annex A-1, Appendix B-11):
http://www.evergladesplan.org/pm/projects/docs_12_decomp_doc_report.aspx

The Dynamic Model for Stormwater Treatment Areas, Version 2 (DMSTA2) was utilized during preliminary screening and final array modeling to confirm compliance with required State of Florida water quality standards.

From initial formulation through selection of the Tentatively Selected Plan (TSP), the CEPP modeling strategy has not included the application of detailed flood event modeling (or hydrodynamic levee assessment). It is expected that higher resolution hydrologic and hydraulic modeling tools will be required to further analyze localized and possibly regional-scale effects of specific components of the CEPP TSP, with the scope of these analyses further identified during the Pre-Construction Engineering and Design (PED) phase of the project.

2. Preliminary Screening

2.1 Summary of Screening Tools and PIR Documentation

Execution of the modeling strategy and application of the initial recommended modeling tool suite initiated in January 2012. Preliminary screening assessments for Lake Okeechobee, the northern estuaries, and the impoundment storage within the EAA, collectively referred to as the "North of the Red Line components," utilized the RESOPS model, the LOOPS model, and the C-43 Spreadsheet Model. The CEPP plan formulation approach, screening methods, and results for the North of Red Line components, which ultimately identified the ~14,000 acre Flow Equalization Basin (FEB) on the EAA A-2 site for inclusion in the CEPP TSP, are summarized in Section 3 of the CEPP draft PIR. Formulation for the FEB component was completed between January and July of 2012. Preliminary screening assessments for the WCAs and ENP, collectively referred to as the "South of the Red Line components," utilized the iModel tool and limited-scope sensitivity simulations using the RSM-GL. The CEPP plan formulation approach, screening methods, and results for the South of Red Line components, which ultimately identified the remaining CEPP TSP components for the L-4/L-5 Levees, Miami Canal, L-67A/L-67C Levees, L-29 Levee, L-67 Extension Levee, and L-31N Canal within WCA-3 and ENP, are summarized in Section 3 of the CEPP draft PIR. Formulation for the Red, Green, Blue, and Yellow Line CEPP components were primarily completed between June and November of 2012. Further documentation of the CEPP screening results and formulation approach is not included in the Engineering Appendix or the supporting Hydrologic Modeling Annex. The CEPP modeling strategy provides an overview of the modeling tools, including maps of the model domains, applied throughout the plan formulation process and how the tools were applied in support of the CEPP planning process (refer to Reference 1).

For the final array of alternatives, analysis of the North of Red Line components and the South of the Red Line components were conducted using the RSM-BN and the RSM-GL, respectively. This H&H Annex provides documentation of USACE SAJ performance analysis of the hydrologic modeling results for the CEPP final array of alternatives only, with specific emphasis on engineering design considerations that were actively tracked throughout the CEPP formulation, preliminary screening, and alternative development efforts. Specific discussion of the performance analysis for the TSP Alternative 4R2 has been summarized within the Engineering Appendix.

2.2 Decomp RMA-2 Screening of Miami Canal Plug Configurations

General overview information and summary conclusions from the Decomp RMA2 screening analysis, which were utilized by the CEPP plan formulation efforts, are documented in this Annex. Summary conclusions from the Decomp analysis were utilized to support CEPP preliminary screening of Miami Canal backfill and plug options. The Decomp RMA2 screening analysis was originally completed by the USACE SAJ Water Resources Engineering Branch between fall 2008 and summer 2009.

The Decomp project conducted a screening model evaluation of numerous Miami Canal plug configurations (plug length and spacing) to identify the optimal configuration(s) which most closely mimic the performance of a complete/full Miami Canal backfill within WCA 3A. The analysis considered both the use of existing fill onsite and importing additional fill to the project from offsite. Due to the limitations of the RMA-2 screening tool, plug configurations were also evaluated with the higher resolution model RSM-GL as part of the final array of alternatives.

Following the CEPP announcement in October 2011, the USACE SAJ and the SFWMD decided to integrate the previous Decomp planning effort into the CEPP. SAJ prepared a documentation Report to summarize the Decomp plan formulation and evaluation efforts, information obtained by the planning team, engineering work efforts, and lessons learned to date. The Decomp documentation report was used by the CEPP team and is available to staff and managers involved in the interagency state-federal Everglades restoration program as a resource to guide future planning efforts. The report documents the plan formulation and evaluation of seven alternatives (subset of final array), all plan formulation activities leading up to the development of the final array of alternatives, recommendations for an adaptive management strategy, and application of extensive hydrologic modeling (including RSM-GL application) conducted to support the formulation and evaluation efforts. The Decomp project documentation report can be reviewed at the following location:

http://www.evergladesplan.org/pm/projects/docs_12_decomp_doc_report.aspx

The Hydrology and Hydraulics Annex to the Engineering Appendix (Annex A-1) of the Decomp documentation report provides comprehensive documentation of the technical support provided by the SAJ Water Resources Engineering Branch: hydrologic data collection and analyses; development and application of numerical modeling tools to support PDT evaluations; preliminary hydraulic design efforts; and additional work-in-progress technical information for consideration by future CERP planning efforts. Annex A-1 of the Decomp PIR 1 project documentation report includes comprehensive documentation of the RMA-2 modeling development methodology and results, and this report should be referenced for additional information, if required.

Recognizing that the Decomp PIR 1 plan formulation process was not completed, the use of the information contained within this Decomp report for future CERP planning efforts included the caveat of recognizing the following key assumptions:

- Alternatives were limited to existing, available water for redistribution within the WCA-3A project area;
- Water quality of inflows to WCA 3A will meet state water quality standards;

- WCA-3A conveyance features to WCA-3B were assumed to be completed and operational (e.g., the Modified Water Deliveries to ENP structures in L-67 A);
- L-29 Canal maximum operating stage limit is 8.5 feet NGVD.

The RMA-2 and RSM-GL modeling efforts conducted for Decomp indicated that plugs along the Miami Canal may have the potential to work as effectively hydrologically as full backfill to reduce drainage and the disruption of sheetflow caused by the Miami Canal. RSM-GL final array modeling during Decomp also revealed that potential benefits from backfilling the Miami Canal south of Interstate 75 were limited under Decomp PIR 1 assumptions (particularly redistribution of existing inflows to WCA-3A only and limited MWD outlet modifications for WCA-3A), probably due to the limited conveyance out of WCA 3A resulting in continued ponded conditions in southern WCA 3A. The 2012 Decomp PIR 1 project documentation report recommended that proposed alterations to the Miami Canal south of Interstate 75 should be reevaluated if the ponding conditions within southern WCA 3A were altered or alleviated.

Although the CEPP TSP proposes significant increased conveyance between WCA-3A, WCA-3B, and ENP as compared to the Decomp assumptions and although the CEPP final array modeling indicates significant reduction to the frequency and magnitude of ponded conditions within southern WCA-3A, no meaningful plan formulation effort was given to modifications to the Miami Canal south of Interstate 75 because the CEPP plan formulation for the WCA-3A hydropattern restoration and Miami Canal components significantly leveraged the previous Decomp formulation efforts. Given consideration of CEPP schedule limitations and based on the results of the CEPP preliminary screening efforts (refer to Section 3.2.2 of the CEPP PIR draft report for detailed discussion of the formulation methodology), CEPP preliminary screening modeling conducted with the RSM-GL in July 2012 evaluated only one option for Miami Canal modifications south of Interstate 75 – inclusion of a 4000 foot long plug centered at S-340 and an 8000 foot long plug starting south of the C-11 Extension. The CEPP RSM-GL screening modeling additionally was conducted as a sensitivity analysis starting with the final array modeling from Decomp with WCA-3A inflows increased to account for the approximately 20 percent increase assumed for CEPP. Therefore, since the CEPP screening modeling assumptions incorporated the MWD project outlet modifications for WCA-3A, the screening modeling results did not demonstrate the expected significant reduction to the frequency and magnitude of ponded conditions within southern WCA-3A that would be realized if the CEPP components identified along the Green Line and Blue Line had been included for the CEPP screening. A different set of CEPP screening assumptions may have demonstrated increased benefits associated with the Miami Canal modifications south of I-75, but these analyses will instead be shifted for future consideration in future CEPP increments.

The plug proposed in the southern reach of the Miami Canal was intended to reduce the drainage effect of the Miami Canal, south of the existing S-340 structure. The Miami Canal south of S-340 and the L-67A Canal currently provides approximately 30 miles of unobstructed southerly canal flow towards the WCA-3A outlet structures along Tamiami Trail (S-333 and the S-12s), and the Miami Canal is aligned parallel to the northwest-to-southeast direction of flow within WCA 3A. In addition, initial screening modeling conducted during Decomp indicated that hydrologic performance improvements within Northeast WCA-3A were generally best achieved through backfill of the South Miami Canal Segment. Effects to recreational access were considered during CEPP formulation of the Miami Canal southern plugs, and the proposed plug location was south of the junction of the Miami Canal/C-11 Extension and north of the

Holiday Trail from Everglades Holiday Park. Recreational access from Everglades Holiday Park to the Miami Canal between S-340 and the proposed plug, to the Miami Canal south of the proposed plug, and to the L-67A Canal will be maintained. Based on review of aerial photographs, the plug length was proposed at 8000 feet, starting south of the C-11 Extension. The source of backfill material for the proposed plug was envisioned as the nearby spoil mounds along the Miami Canal and the then CEPP-proposed spoil mound degrade/gaps along the C-11 Extension (this component was subsequently excluded with the CEPP final array), with the proximity of the C-11 Extension spoil material serving as a factor in the plug location selection. Based on preliminary surveys of the Miami Canal spoil mound material under Decomp, a maximum of approximately 5.5 of the 9.7 miles (57%) of the Miami Canal between S-340 and the L-67A Canal could be backfilled with the on-site spoil material.

The Miami Canal plug configuration was ultimately screened out from the CEPP final array components because the RSM-GL screening modeling demonstrated only localized dry year benefits for the single evaluated plug configuration, which could not justify the additional incremental cost of approximately $13 million. However, consistent with the Decomp report conclusions, the final conclusions identified from the CEPP screening assessment should include consideration of the assumptions related to limited relief for the ponding conditions in southern WCA-3A and the limited spatial extent of plugs which were evaluated. Given recognition of this context, consideration of Miami Canal modifications south of Interstate 75 will likely warrant further detailed evaluation for future CERP/CEPP increments.

Beyond the insights afforded by hydrologic modeling, as further summarized in the Decomp report, questions remain regarding the ability of plugged canals to function ecologically as the pre-drainage ridge and slough landscape, especially in low flow conditions, and what the continuing effect of deep holes (spaces between plugs) in the canal have on Everglades flora and fauna, including providing pathways for invasive exotic species. These uncertainties would need further assessment for consideration of future plug options for canals within the Greater Everglades, although additional information may also be realized through CEPP adaptive management strategies.

Although the Miami Canal plugs were not included in the components for the CEPP final array (all final array alternatives included complete backfill of the Miami Canal to Interstate 75, starting from either approximately 1.5-2.0 miles south of S-8 (Alternative 1, 4R, and 4R2) or immediately downstream of S-8 (Alternatives 2 through 4)), information from the Decomp RMA-2 plug analysis was additionally utilized to establish the initial proposed spacing between Miami Canal mounds because the Miami Canal backfill to bedrock grade will leave remnant open water segments between the mounds that are expected to behave hydrologically similar to the plug options that were evaluated with RMA-2 for Decomp. As further documented in Table 3 and Table 4, overall plug performance (compared to the full backfill condition) is significantly diminished for plug spacing scenarios greater than approximately 4000-6000 feet, whereas no observed similar trend is observed for plug length. The initial proposed spacing between Miami Canal mounds was selected at 1 mile (5280 feet), given consideration of the insights from the Decomp RMA-2 modeling and overall CEPP project cost considerations (increased cost with reduced distance between mound features).

The Decomp modeling strategy recommended a limited modeling effort, utilizing a fine resolution hydraulic modeling tool, to allow evaluation of the potential near-field effects for

Miami Canal backfill options and yield enhanced understanding about the effectiveness and impacts of each type of canal backfilling option. The need to simulate three dimensional flow fields was not a critical element of the Miami Canal local feature modeling effort; two-dimensional flow fields with depth-averaged velocity parameters (including within open canal segments) were determined to provide sufficient analysis for the stated scope of this effort, noting the shallow depths representative of typical overland flow in the project area. RMA2 was recommended by the USACE within the Decomp modeling strategy as the most appropriate tool for this analysis. RMA2, developed by the Resource Management Associates (RMA), has been previously reviewed and classified by the HH&C CoP as a “CoP Preferred” hydraulic design and river hydraulics modeling tool.

RMA2 is a two dimensional depth averaged finite element hydrodynamic numerical model. It computes water surface elevations and horizontal velocity components for subcritical, free-surface flow in two dimensional flow fields. RMA2 computes a finite element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows. Friction is calculated with the Manning’s or Chezy equation, and eddy viscosity coefficients are used to define turbulence characteristics; RMA2 includes the capability to vary roughness parameters as a function of flow depth. Both steady and unsteady state (dynamic) problems can be analyzed. RMA2 accepts a wide variety of boundary conditions including water surface elevations by node or line, discharge by node, element or line, tidal radiation boundary conditions by line and discharge as a function of elevation by line. The program has been applied to calculate water levels and flow distribution around islands; flow at bridges having one or more relief openings, in contracting and expanding reaches, into and out of off-channel hydropower plants, at river junctions, and into and out of pumping plant channels; circulation and transport in water bodies with wetlands; and general water levels and flow patterns in rivers, reservoirs, and estuaries. RMA2 operates under the hydrostatic assumption, meaning accelerations in the vertical direction are negligible. It is two dimensional in the horizontal plane. It is not intended to be used for near field problems where vortices, vibrations, or vertical accelerations are of primary interest. Vertically stratified flow effects are beyond the capabilities of RMA2. RMA2 is a free-surface calculation model for subcritical flow problems.

As presented to the Decomp PDT in July 2008, the following were the goals of the Decomp RMA2 modeling effort:

- Utilize available data to construct a fine-resolution hydraulic model to test Miami Canal backfill options, within a limited spatial scale, representative of the project area;
- Evaluate near-field hydraulic effects of various plug designs, lengths, spacing, locations, and depths (partial vs. complete fill) to gain additional information to assist project team evaluation of Miami Canal backfill options;
- Provide additional input to assist preliminary screening and determine optimal use of limited fill material available with the existing spoil mounds;
- Balance needs of the project schedule while maximizing the value-added from effective model simulation of backfill options.

The Decomp H&H sub-team advocated for use of the Miami Canal segment between S-8 and S-339 as reasonably representative of the Miami Canal hydrologic response and recommended a series of steady-state simulations for each evaluated Miami Canal backfill option. Topography for the RMA2 mesh was assigned from Decomp PIR 1 Miami Canal survey data, available Light

Detection and Ranging (LIDAR) data, and United States Geological Survey (USGS) High-Accuracy Elevation Data (HAED). The RMA2 model domain from S-8 to S-339, overlaid on an aerial photograph, is shown in Figure 1; the model domain is approximately 9.5 miles in length (along the Miami Canal) and approximately 4 miles in width (approximately 2 miles on each side of the Miami Canal). The model mesh elements are on the scale of 20 to 40 feet in length in the immediate vicinity of the Miami Canal and approximately 550 feet in length for elements within approximately 2 miles of the Miami Canal. The steady-state RMA2 model simulations utilized an upstream point-source inflow boundary condition at S-8 and a downstream head boundary condition along the southern model boundary (representative of S-339). The median flow rate for the S-8 pump station between 2000 and 2008 was used to define the upstream steady-state flow boundary condition of 950 cfs; arithmetic average historical stage predictions from the USGS Everglades Depth Estimation Network (EDEN) at four points along the southern model boundary, during all 2000-2008 conditions with S-8 historical flows approximating 950 cfs (a range of 900-1000 cfs was used), were used to establish the steady-state downstream head boundary condition of 10.66 feet NGVD at S-339 (approximate depth of 1 foot in the adjacent WCA-3A marsh). The S-339 structure along the Miami Canal was closed for all simulations, consistent with field operations of S-339 for this flow range. The RMA2 model was not needed to evaluate a wide range of hydrologic conditions or to assess the effects of groundwater or seepage, as the RSM-GL model would be relied on during the later Decomp alternative modeling to assess the performance of Decomp management measures (including Miami Canal plugs) for the 1965 through 2000 period of record based on the results of the RMA-2 screening analysis.

Due to the large potential number of Miami Canal plug length and spacing combinations and the challenges experienced by the Decomp PDT identifying only approximately three combinations for detailed RMA2 modeling (as originally scoped), the RMA2 modeling scope evolved into a robust modeling assessment of a wide range of Miami Canal plug length and spacing combinations. Since each Miami Canal plug length and spacing combination required the development of a distinct RMA2 model mesh, a sufficient number of the potential combinations were simulated to generate nomographs (refer to Figure 2 for a generalized example) able to illustrate performance trends of the simulated plug length and spacing combinations and allow interpolation of expected performance for non-simulated combinations. Table 1 illustrates the five Miami Canal plug lengths (500 feet to 10000 feet) and the eight Miami Canal plug spacing (1000 feet to 25000 feet) that were used to establish the RMA2 model test matrix; 18 total plug length and spacing combinations were simulated, in addition to the existing condition (no Miami Canal modifications), the existing condition with all Miami Canal spoil mounds removed (reasonable initial step toward sheetflow restoration), and the full backfill condition (complete Miami Canal backfill between S-8 and S-339, used to establish a target for sheetflow conditions). To represent Miami Canal plugs within the RMA2 model, the nodal elevations within the specified Miami Canal plug footprint were changed to match the adjacent marsh. For each plug scenario, a 1000 foot segment of the Miami Canal was left unmodified to provide a hydraulic get-away for the S-8 inflows to the model domain and all Miami Canal spoil mounds were assumed to be removed (reasonably expected consistent with Decomp sheetflow restoration objective).

The test matrix simulations were developed to answer the following question: for a given plug length, what is the optimal plug spacing to mimic the total backfill case? In order to maximize the utility of the RMA2 analysis for the Decomp alternative formulation process, the Decomp H&H sub-team developed and agreed on a set of post-processing and statistical analysis performance metrics to facilitate evaluation of the RMA2 results; unlike the SFWMM and RSM Glades-LECSA model, a pre-established set of performance measures is not generated from RMA-2. The following performance measures were evaluated for each RMA2 test matrix simulation, compared to the total backfill case:

- Correlation Coefficient of absolute velocity magnitude
- Correlation Coefficient of along canal velocity
- Correlation Coefficient of across canal velocity
- Cosine Similarity (velocity vector)
- Flow across transect

The correlation coefficient, R, determines the strength of a relationship between two datasets. Cosine similarity is a measure of similarity between two vectors of n dimensions by finding the cosine of the angle between them. The selected flow transect was established approximately 2 miles upstream from S-339 and extended approximately 100 feet on both sides of the Miami Canal. All of the performance metrics for each plug length and spacing configuration were computed by comparison against the total backfill case.

The preliminary performance measure results are summarized in Table 2. Following review of the results by the Decomp H&H sub-team, the correlation coefficient of across canal velocity and the cosine similarity performance measures were dropped due to the lack of any trend or correlation to the plug configuration. The three remaining performance measures were ranked for each of the 18 plug configurations (1 = best compared to total backfill case; 18 = worst compared to total backfill case) and the scores for each performance measure were added together to produce a final combined score for each plug configuration; the complete results are displayed in Figures 3 through 5, and Tables 2 and 3. The summary for the top six plug configurations are displayed in Table 4.

The final RMA2 modeling results were presented to the Decomp PDT in July 2009. The RMA-2 conceptual plug design analysis results were then available to the Decomp plan formulation team to be integrated with the preliminary engineering costs for construction of the proposed Miami Canal management measures, PDT assessment of on-site fill availability (spoil mound material quantities, acceptability of spoil mound and associated vegetation/upland habitat removal, and material suitability for backfill), proposed construction methods for Miami Canal modifications, and the results from the SFWMM preliminary screening of Miami Canal management measures (refer to the Decomp PIR 1 project documentation report for additional documentation of the formulation process).

As shown in the last column of Table 3, each plug configuration was also able to be evaluated against the available volume of fill required. Based on preliminary USACE design calculations based on the Miami Canal survey data, approximately 2.66 million cubic yards (MCY) of potential backfill material is available in the existing Miami Canal spoil mounds. Approximately 4.52 MCY of material would be required to backfill the entire 27 miles of the Miami Canal within the Decomp PIR 1 project footprint (S-8 to S-151). Based on consideration of this material shortfall for a complete backfill, there is a recognized cost associated with importing the additional fill needed for any plug configuration that exceeds the 2.66 MCY estimated available on site; the final combined score (total ranking) for each plug configuration is plotted against the fill volume required to replicate each individual plug configuration along the entire 27 mile

Miami Canal project area length in Figure 6. Based on the RMA2 performance measure evaluation, the plug configuration with a 4000 foot plug length and 2000 foot spacing provided the closest match to the total backfill case, although this configuration, in the absence of further optimization (i.e. slightly shorter plug length or slightly increased plug spacing, not likely to significantly affect overall performance), would require the import of an estimated 0.35 MCY of material in excess of the spoil material available on site. If no additional fill material is generated from other components of Decomp PIR 1 (spreader canal construction, canal conveyance improvements, etc.) and fill availability is utilized as a selection criteria for the optimal plug configuration, the 1000 foot plug length and 3000 foot spacing would represent the preferred plug configuration.

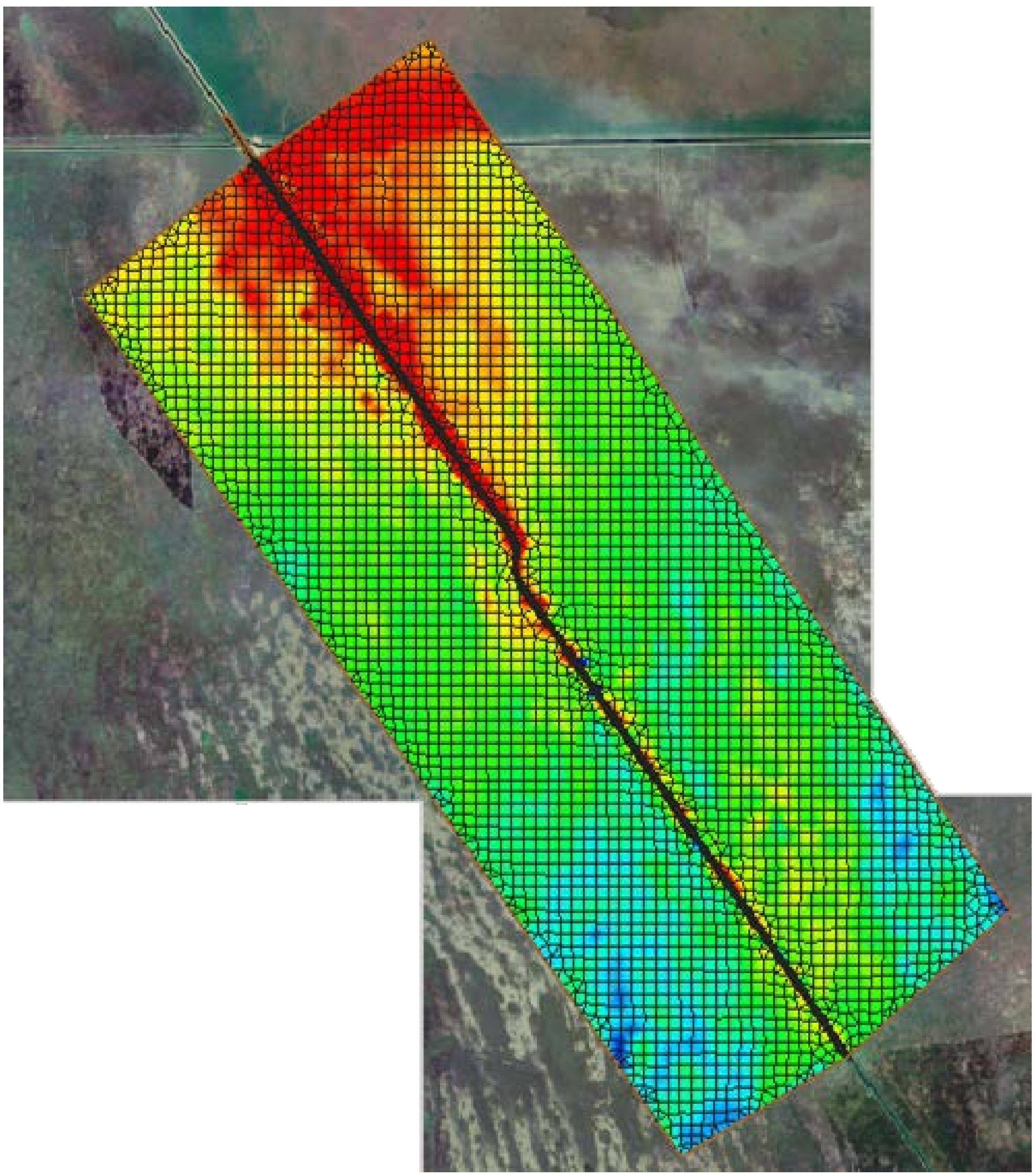


Figure 1: RMA2 Model Domain for Screening of Miami Canal Plug Configurations

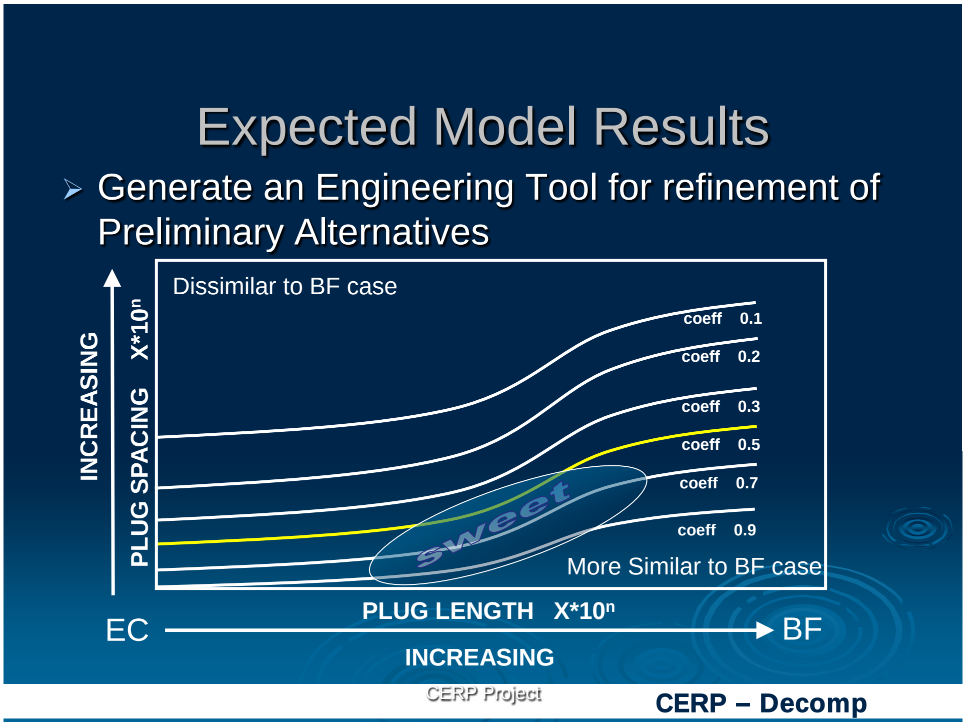


Figure 2: Example of RMA2 Analysis Approach for Screening of Miami Canal Plug Configurations

Table 1: RMA2 Model Test Matrix, Miami Canal Plug Length and Spacing Combinations Evaluated

Length (feet)	500	1000	2000	4000	10000
Spacing (feet)					
1000	X		X	X	
3000	X	X			
4000			X		
5000	X	X			X
10000		X	X	X	X
15000	X				
20000			X	X	
25000	X	X			

Table 2: Preliminary RMA2 Performance Measure Screening Results for Miami Canal Plug Configurations

Treatment # (plug Configuration)	Plug Length	Plug Spacing	R Velocity Magnitude	Cosine Similarity	R Velocity Along Canal	R Velocity Across Canal	Transect Flow (cfs)	Normalized Transect Flow (30 cfs for total backfill case)
1	**500**	**1000**	**0.96**	**0.82**	**0.94**	**0.86**	**49**	**1.6**
2	**500**	**3000**	**0.86**	**0.81**	**0.84**	**0.82**	**70**	**2.3**
3	**500**	**6000**	**0.81**	**0.81**	**0.79**	**0.80**	**120**	**4.0**
4	**500**	**12000**	**0.65**	**0.82**	**0.63**	**0.79**	**171**	**5.7**
5	**500**	**24000**	**0.57**	**0.84**	**0.53**	**0.80**	**197**	**6.6**
6	**1000**	**3000**	**0.96**	**0.82**	**0.95**	**0.89**	**55**	**1.8**
7	**1000**	**5000**	**0.86**	**0.82**	**0.84**	**0.86**	**83**	**2.8**
8	**1000**	**12000**	**0.67**	**0.83**	**0.65**	**0.84**	**176**	**5.9**
9	**1000**	**23000**	**0.58**	**0.84**	**0.55**	**0.86**	**193**	**6.4**
10	**2000**	**1000**	**0.98**	**0.84**	**0.97**	**0.94**	**47**	**1.6**
11	**2000**	**4000**	**0.93**	**0.82**	**0.92**	**0.90**	**107**	**3.6**

Treatment # (plug Configuration)	**Plug Length**	**Plug Spacing**	**R Velocity Magnitude**	**Cosine Similarity**	**R Velocity Along Canal**	**R Velocity Across Canal**	**Transect Flow (cfs)**	**Normalized Transect Flow (30 cfs for total backfill case)**
12	**2000**	**10000**	**0.75**	**0.83**	**0.72**	**0.88**	**164**	**5.5**
13	**2000**	**23000**	**0.62**	**0.85**	**0.59**	**0.89**	**185**	**6.2**
14	**4000**	**2000**	**0.98**	**0.86**	**0.98**	**0.97**	**38**	**1.3**
16	**4000**	**10000**	**0.88**	**0.84**	**0.86**	**0.93**	**93**	**3.1**
16	**4000**	**18000**	**0.71**	**0.85**	**0.69**	**0.93**	**150**	**5.0**
17	**10000**	**5000**	**0.90**	**0.85**	**0.80**	**0.95**	**189**	**6.3**
18	**10000**	**10000**	**0.89**	**0.85**	**0.88**	**0.97**	**123**	**4.1**

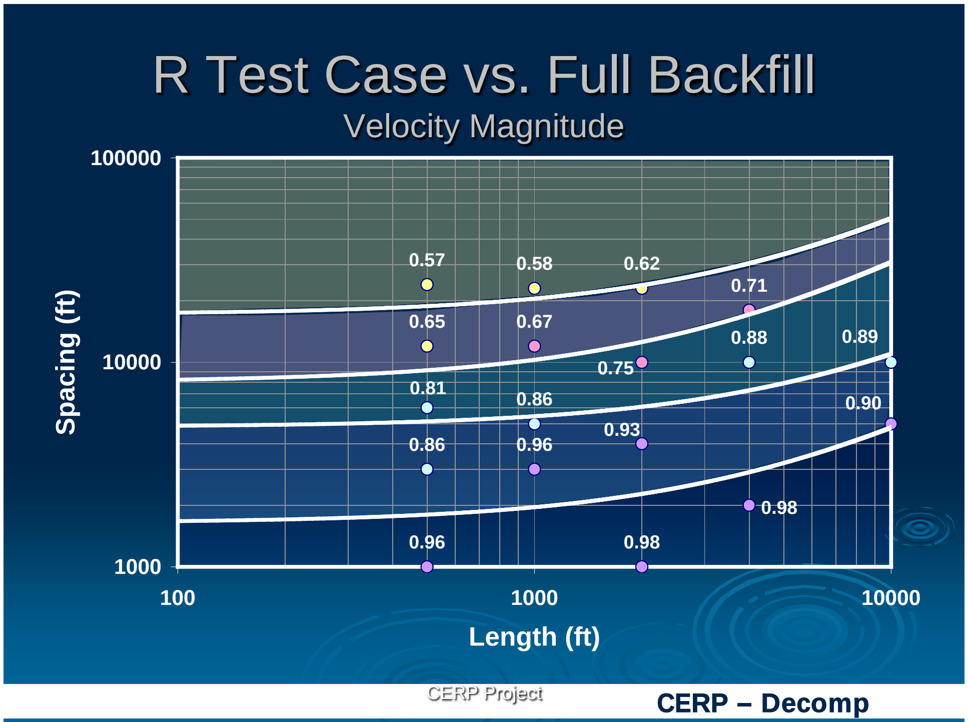


Figure 3: RMA2 Performance Measure Screening Results, Correlation Coefficient of absolute velocity magnitude

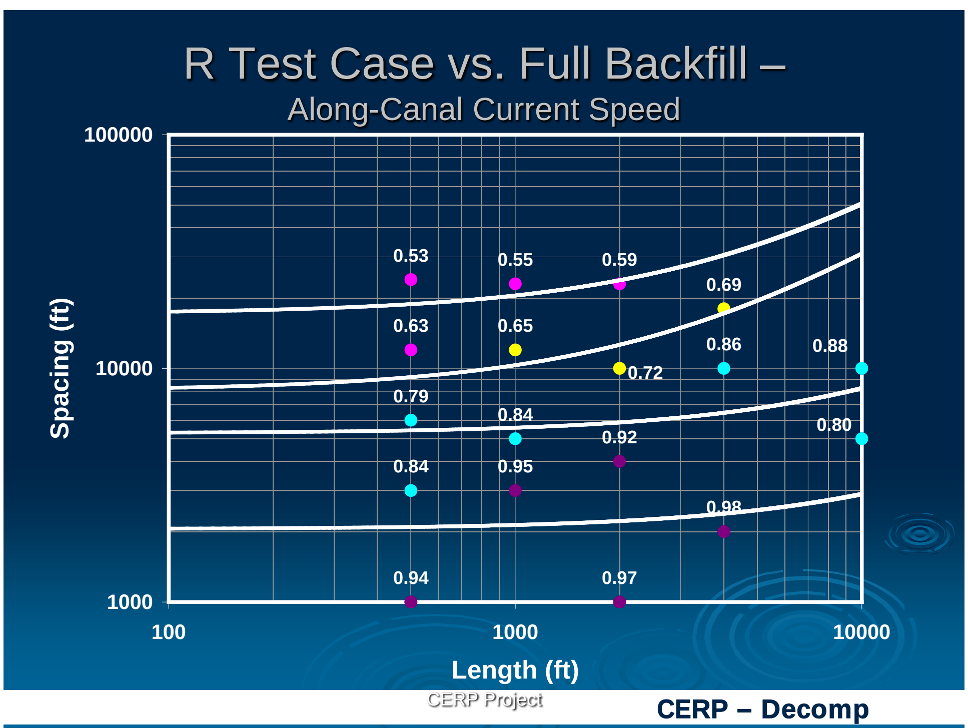


Figure 4: RMA2 Performance Measure Screening Results, Correlation Coefficient of along canal velocity

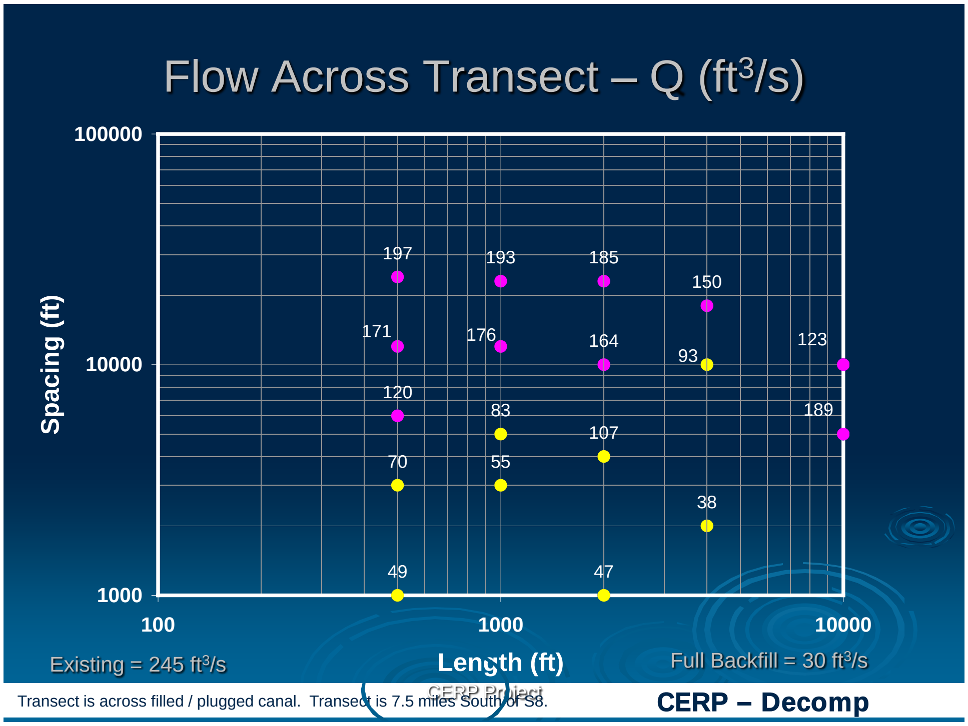


Figure 5: RMA2 Performance Measure Screening Results, Flow across transect

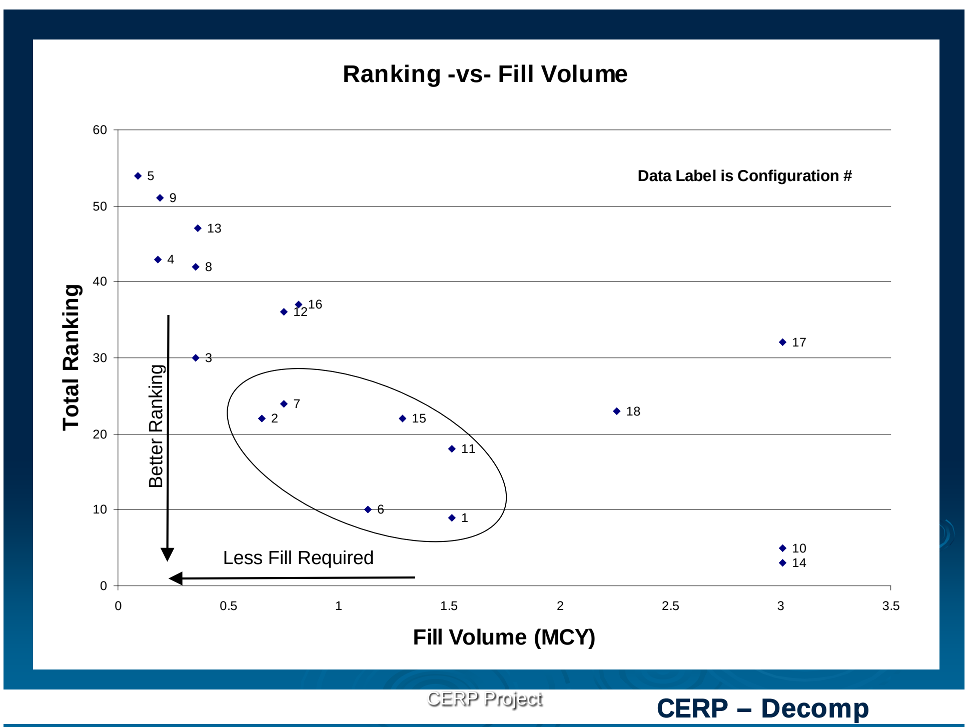


Figure 6: RMA2 Performance Measure Screening Results, Combined Score as a Function of Plug Fill Requirements

Table 3: Final RMA2 Performance Measure Screening Results for Miami Canal Plug Configurations

Treatment # (plug Configuration)	Plug Length	Plug Spacing	R Velocity Magnitude Rank	R Velocity Along Canal Rank	Transect Flow Rank	Combined Score	Fill Required (MCY)
1	**500**	**1000**	3	4	2	9	1.51
2	**500**	**3000**	9	8	5	22	0.65
3	**500**	**6000**	10	11	9	30	0.35
4	**500**	**12000**	15	15	13	43	0.18
5	**500**	**24000**	18	18	18	54	0.09
6	**1000**	**3000**	3	3	4	10	1.13
7	**1000**	**5000**	10	8	6	24	0.75
8	**1000**	**12000**	14	14	14	42	.035
9	**1000**	**23000**	17	17	17	51	0.19
10	**2000**	**1000**	1	2	2	5	3.01
11	**2000**	**4000**	5	5	8	18	1.51

Treatment # (plug Configuration)	Plug Length	Plug Spacing	R Velocity Magnitude Rank	R Velocity Along Canal Rank	Transect Flow Rank	Combined Score	Fill Required (MCY)
12	**2000**	**10000**	12	12	12	36	0.75
13	**2000**	**23000**	16	16	15	47	0.36
14	**4000**	**2000**	1	1	1	3	3.01
15	**4000**	**10000**	8	7	7	22	1.29
16	**4000**	**18000**	13	13	11	37	0.82
17	**10000**	**5000**	6	10	16	32	3.01
18	**10000**	**10000**	7	6	10	23	2.26

Table 4: Final RMA2 Performance Measure Screening Results for the Top 6 Ranked Miami Canal Plug Configurations

Configuration #	Plug Length	Plug Spacing	Combined Score	Fill Volume Required (MCY)
14	4000	2000	3	3.01
10	2000	1000	5	3.01
1	500	1000	9	1.51
6	**1000**	**3000**	**10**	**1.13**
11	2000	4000	18	1.51
2	500	3000	22	0.65

*Available fill onsite = ~2.66 MCY

3. Evaluation of the Final Array of Alternatives

3.1 CEPP Baseline Condition Modeling

The study area for the CEPP encompasses Lake Okeechobee, the Northern Estuaries (St. Lucie River and Indian River Lagoon and the Caloosahatchee River and Estuary), a portion of the EAA, the WCAs, ENP, the Southern Estuaries (Florida Bay and Biscayne Bay), and the Lower East Coast. Section 2.4 of the CEPP PIR main report provides a summary description of the existing and future without project conditions within the study area. Detailed documentation of existing and future without project conditions is further provided in Appendix C.1 to the CEPP PIR main report, including detailed documentation of hydrology, regional water management, flood control, and water supply performance for each base condition. Selected graphics are included to illustrate the performance of each baseline condition.

Hydrologic modeling simulations of the existing condition baseline (ECB) and the CEPP future without project condition (FWO) were developed with the RSM-BN and RSM-GL sub-regional modeling tools, to provide baseline conditions for plan formulation and the assessment of CEPP project benefits and the assessment of CEPP alternative performance for the level-of-service for flood protection and water supply (ECB). The ECB was developed to represent the system-wide infrastructure and operations that were in place at the time CEPP plan formulation was initiated, approximately January 2012. The FWO for CEPP assumes the construction and implementation of currently authorized C&SF and non-CERP projects, and other Federal, state or local projects constructed or approved under existing governmental authorities that occur in the CEPP study area; the CEPP FWO therefore included first generation CERP projects already authorized and under construction (Indian River Lagoon-South Project, Picayune Strand Restoration Project, Site 1 Impoundment Project), second generation CERP projects still pending Congressional authorization (Biscayne Bay Coastal Wetlands Project, Broward County Water Preserve Areas Project, Caloosahatchee River (C-43) West Basin Storage Reservoir, C-111 Spreader Canal Western Project), and non-CERP projects currently in progress (SFWMD Restoration Strategies, C&SF C-51 West End Flood Control Project, the C-111 South Dade Project, the Kissimmee River Restoration Project, Modified Water Deliveries, and the DOI Tamiami Trail Modifications Next Steps Project. Documentation of RSM-BN and RSM-GL assumptions for the ECB and FWO baseline conditions are provided in Reference 2 of this Annex, respectively.

The CEPP PIR report documentation and two complete sets of RSM-BN and RSM-GL hydrologic model performance measure output are posted on the Everglades Plan public web site for the CERP:
http://www.evergladesplan.org/pm/projects/proj_51_cepp.aspx

The following complete performance measure data sets are provided to facilitate additional review of the hydrologic modeling output for the baselines and the TSP Alternative 4R2:

- ECB, FWO, Alternative 4R, Alternative 4R2 (comparison used for NEPA evaluation in Section 5 of the main PIR report)
- ECB, 2012EC, IORBL1,Alternative 4R2 (comparison used for the Savings Clause and Project Assurances evaluation in Annex B of the PIR report)

3.2 Final Array Modeling

CEPP plan formulation efforts identified the final array of four alternatives (Alternatives 1 through 4) in November 2012, and the corresponding RSM-BN and RSM-GL simulations of the alternatives was subsequently completed in December 2012. As documented in Section 4.6 of the CEPP PIR main report, modifications to the final array were identified during January-February 2013 that resulted in the identification of Alternative 4M as the National Ecosystem Restoration (NER) Plan. The evaluation also identified the need to revise the operations for Alternative 4M, which was not evaluated with hydrologic modeling, to ensure the project savings clause constraints are met, to minimize localized adverse ecological effects, and to identify additional opportunities to provide for other water related needs.

Three additional modeling scenarios were conducted in the following months to identify project effects resulting from the identified operational changes: Alternative 4R (completed February 2013), Alternative 4R1 (May-June 2013), and Alternative 4R2 (June 2013). The first refinement, Alt 4R, focused on operation changes to avoid potential impacts to water supply levels of service in the Lake Okeechobee Service Area (LOSA) and Lower East Coast (LEC). Refinements included alleviating potential ecological impacts from lowered water depths in WCA 2B by retaining a small portion of the water in WCA 2B that Alternative 4M had diverted to WCA 3A. Increases in low flow events to the St. Lucie Estuary, minimized reductions in freshwater flows to Biscayne Bay, and improved water depths in eastern WCA 3B for purposes of improving environmental conditions were also considered. Building on the performance improvements achieved with the Alternative 4R operational changes, Alternatives 4R1 and 4R2 increased public water supply demand for Lower East Coast Service Area 2 (LECSA 2 - Broward County) and Lower East Coast Service Area 3 (LECSA 3 - Miami-Dade County) to determine whether there was a threshold for increased public water supply demand that would be capable of balancing increased water supply demands for LECSA 2 and LECSA 3 with maintaining the natural system performance of Alternative 4R. Alternative 4R1, which increased public water supply demand by 19 million gallons per day (MGD) for LECSA 2 and 53 MGD for LECSA 3, was not assessed in detail in the PIR report due to significant performance concerns identified with the observed reductions in discharges to Biscayne Bay and increased risk of saltwater intrusion at several wellfield locations. Based on information gained during the modeling of Alternative 4R1 and related RSM-GL sensitivity simulations, the subsequent Alternative 4R2 simulation limited the increase to public water supply demand by 12 MGD for LECSA 2 and 5 MGD for LECSA 3 and was determined to be successful with maintaining the ecological performance of Alternative 4R without the negative effects to LEC groundwater and Biscayne Bay that Alt 4R1 realized. Alternative 4R2 was identified in the PIR main report as the TSP Plan.

Completion of the model documentation reports for the model assumptions was deferred to following completion of the CEPP final array and Project Assurances/Savings Clause modeling (this information will be provided as additional supporting documentation for the Final PIR). Prior to the availability of the complete model documentation reports, model assumption tables for all alternatives analyzed in the PIR main report are provided in Reference 2 of this Annex.

The study area for the CEPP encompasses Lake Okeechobee, the Northern Estuaries (St. Lucie River and Indian River Lagoon and the Caloosahatchee River and Estuary), a portion of the EAA, the WCAs, ENP, the Southern Estuaries (Florida Bay and Biscayne Bay), and the Lower East Coast. Section 4 and Section 5 of the CEPP PIR main report provides a performance evaluation for the final array of alternatives. Detailed documentation of the effects of the alternatives 1 through 4 on regional hydrology and water supply and flood control, compared to the future without project base condition, are provided in Section 5.1.8, Section 5.1.15.2 and Appendix C.2.1 of the CEPP PIR main report. Detailed documentation of the effects of the operational refinements of the TSP plan (Alternative 4R and Alternative 4R2) on regional hydrology and water supply and flood control, compared to the future without project base condition, are provided in Sections 5.2.8, Section 5.2.15.2, and Appendix C.2.2 of the CEPP PIR main report. Selected graphics are included to illustrate the performance of each alternative.

An enormous amount of output is generated from each RSM-BN and RSM-GL simulation and the accompanying post-processed performance measures. Reference maps to assist with user navigation of RSM-GL indicator regions, performance measure zones, transects, reference gages, and viewing window spatial locations are included in Reference 3 of this Annex. The monitoring gage map, levee map, and transect map are additionally included in this Annex as Figures 7 through 9.

For the CEPP, standard RSM-BN and RSM-GL performance measure output are grouped into the following directory structure, to assist with user navigation: duration curves; Florida Bay salinity (directory name: FlaBay_Salinity); flow magnitude (data files summarize daily cosine similarity statistics for surface water flow vectors); Lake Okeechobee; Northern Estuaries; percent period-of-record inundation for ridge and slough landscape (PPOR inundation); slough vegetation; soil oxidation; continuity metric for transects (transectflow_continuity); distribution metric for transects (transectflow_distribution); and timing metric for transects (transectflow_timing). For the CEPP, standard RSM-BN and RSM-GL performance indicators output are grouped into the following directory structure, to assist with user navigation: critical flow (data file summarizes average annual structure flows); 1983-1993 stage duration curves for selected indicator cells within the Lower East Coast (duration_8393); water restriction frequency for Lower East Coast Service Areas (freq_water_restrictions); hydrographs and stage duration curves for selected recreational camp locations (hyd_dur_camps); hydrographs and stage duration curves for C&SF canals (hyd_dur_canals); hydrographs and stage duration curves for selected monitoring gages within the Greater Everglades and Lower East Coast (hyd_dur_gages); hydrographs and stage duration curves for Lake Okeechobee (Lake Okeechobee); Lake Okeechobee Service Area water supply performance, including Seminole Tribe reservations (LOSA_Water_Supply); Northern Estuaries; Lower East Coast levee seepage (seepage_reports); average annual wet and dry season transect flows (transect_flows); and Everglades water level spatial and temporal variability viewing windows (viewing_windows). The performance indicators directory also includes sub-directories for each baseline condition and alternative, which include annual average and period-of-simulation average graphics for groundwater vector maps, hydroperiod maps, ponding depth maps, stage maps, surface water vector maps, water budget maps (period-of-simulation average only), and average April water stage level maps for the Lower East Coast (average year 1978, dry years 1989 and 2001, and wet year 1995).

The CEPP final array modeling output includes two performance measure sets that include: (1) concurrent performance measure display of the CEPP FWO outputs and Alternative 1 through 4, including combined outputs for both the RSM-BN and RSM-GL models; and (2) concurrent performance measure display of the CEPP FWO outputs, Alternative 4R, and Alternative 4R2, including combined outputs for both the RSM-BN and RSM-GL models.

The CEPP PIR report documentation and two complete sets of RSM-BN and RSM-GL hydrologic model performance measure output are posted on the Everglades Plan public web site for the CERP:
http://www.evergladesplan.org/pm/projects/proj_51_cepp.aspx

The following complete performance measure data sets are provided to facilitate additional review of the hydrologic modeling output for the baselines and the TSP Alternative 4R2:

- ECB, FWO, Alternative 4R, Alternative 4R2 (comparison used for NEPA evaluation in Section 5 of the main PIR report)
- ECB, 2012EC, IORBL1,Alternative 4R2 (comparison used for the Savings Clause and Project Assurances evaluation in Annex B of the PIR report)

For additional reference, the following selected RSM-BN and RSM-GL is provided within this Hydrologic Modeling Annex. Reference 4 of this Hydrologic Modeling Annex includes the RSM-BN water budget map output for the CEPP baselines (ECB, 2012EC, FWO, and IORBL1) and Alt4R2, including structure crosswalk information for the water budget maps. Reference 5 of this Hydrologic Modeling Annex includes the RSM-GL water budget map output for the CEPP baselines (ECB, 2012EC, FWO, and IORBL1) and Alt4R2. Reference 6 of this Hydrologic Modeling Annex includes the RSM-GL stage output maps, RSM-GL hydroperiod output maps, stage difference maps for Alt4R2 compared to each baseline, and hydroperiod difference maps for Alt4R2 compared to each baseline.

Sections 3.2.1 through 3.2.4 of this H&H Annex provide documentation of USACE SAJ performance analysis of the hydrologic modeling results for the CEPP final array of alternatives with specific emphasis on engineering design considerations that were actively tracked throughout the CEPP formulation, preliminary screening, and alternative development efforts. Following the CEPP PDT evaluations of Alternatives 1 through 4, Alternative 4 was the alternative selected for further optimization and which ultimately became the TSP plan, Alternative 4R2. Since the modeling for Alternative 4R and Alternative 4R2 was conducted subsequent to the modeling for Alternatives 1 through 4, most figures within the Annex are duplicated for: (1) Alternatives 1 through 4; and (2) Alternatives 4R and 4R2 only (Alternative 4 may also be shown, as an additional reference point). Within the Engineering Appendix, summary information is typically provided only for the TSP, Alternative 4R2.

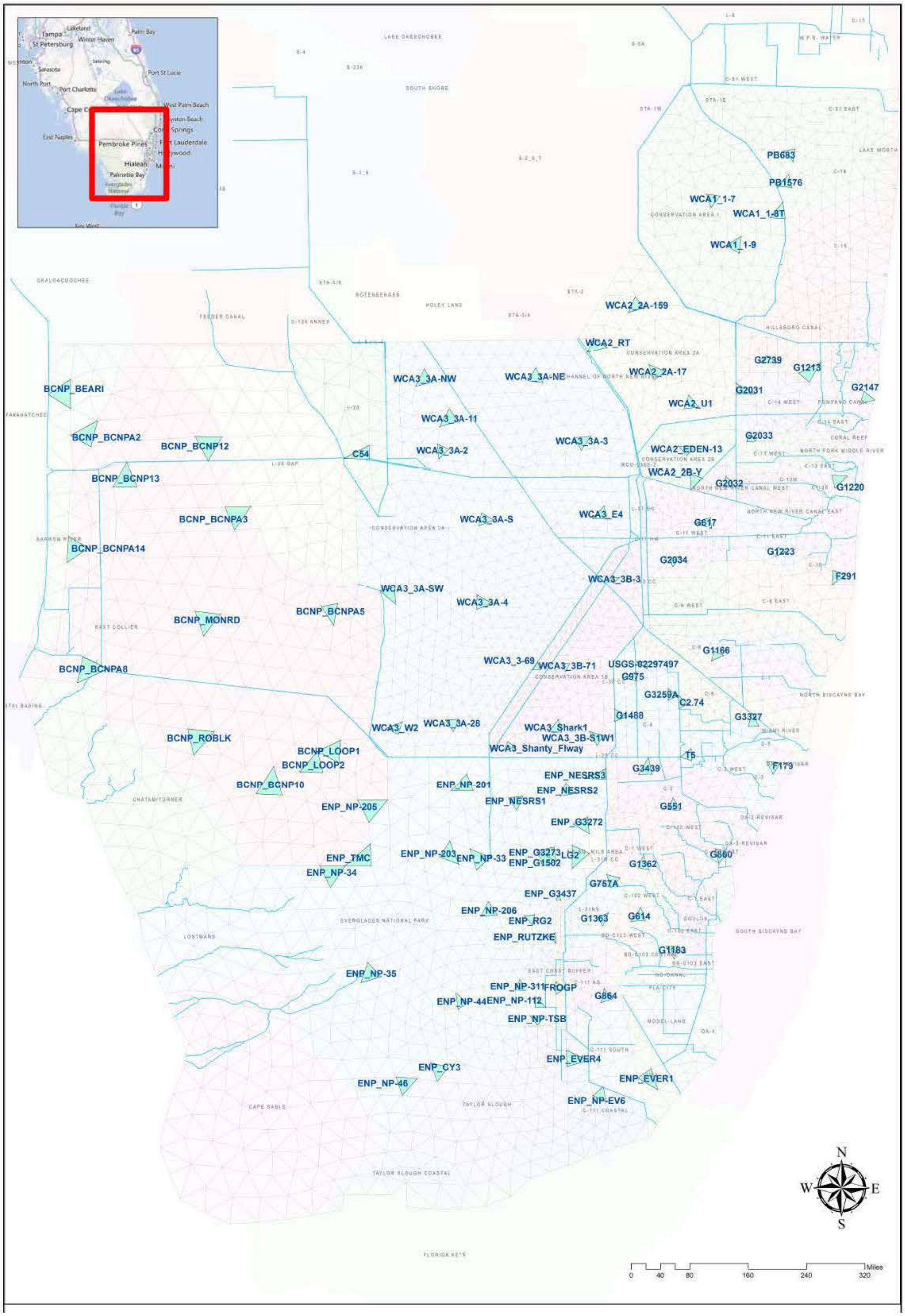


Figure 7: Map of RSM-GL monitoring gage locations

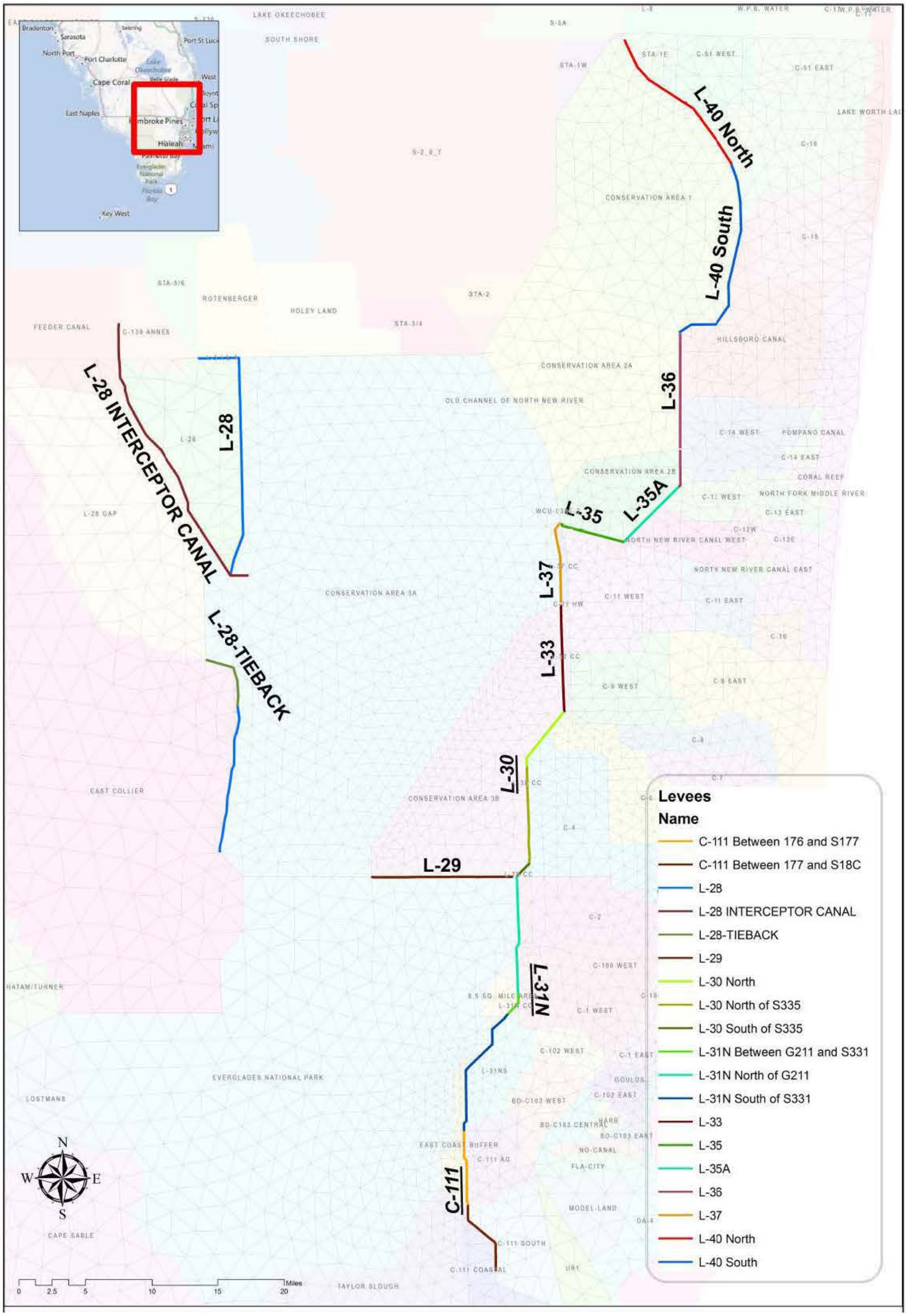


Figure 8: Map of RSM-GL perimeter seepage levees

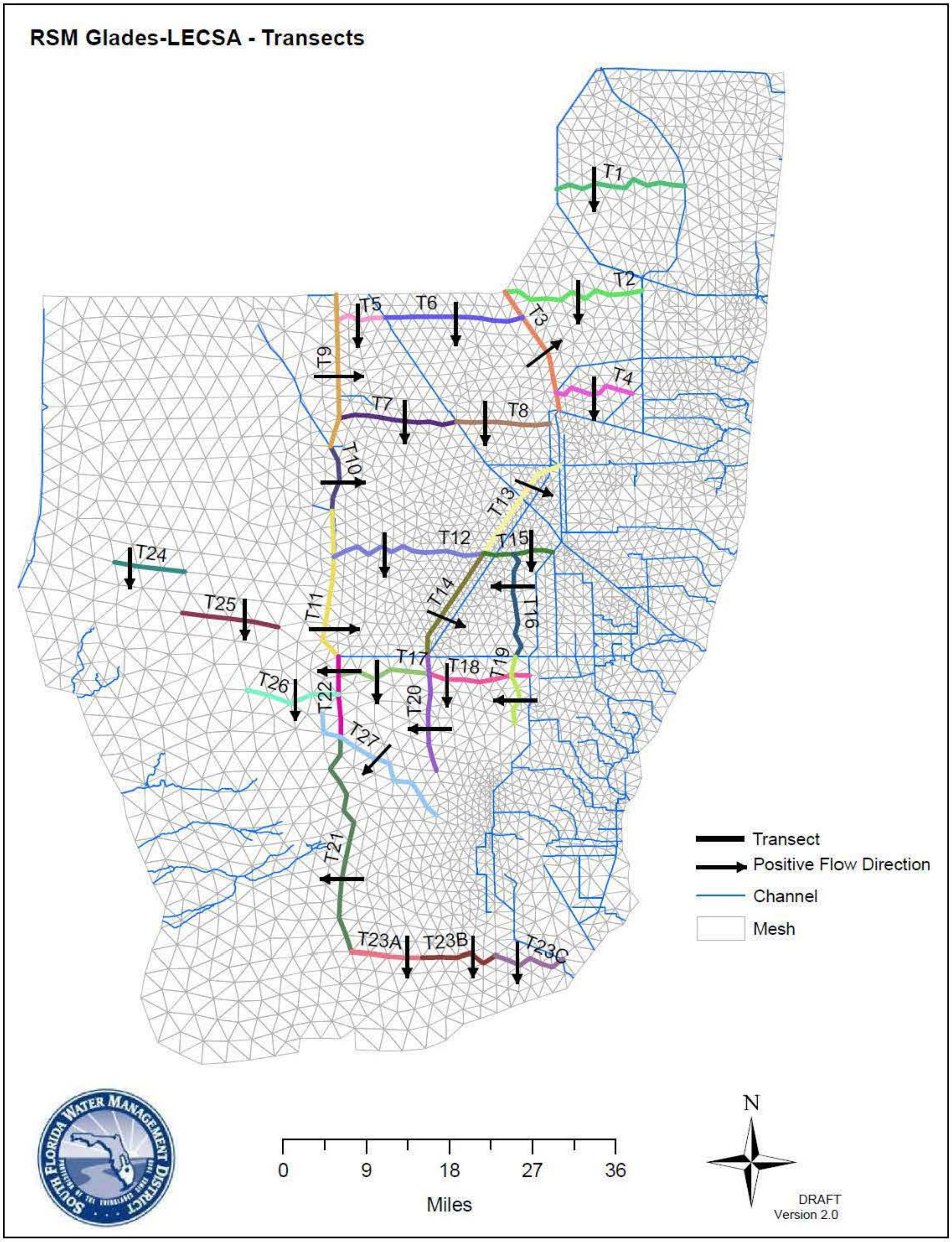


Figure 9: Map of RSM-GL surface water and groundwater reporting transects

3.2.1. WCA-3A High Water Performance Evaluation Methodology and Results

The USACE Final Everglades Restoration Transition Plan (ERTP) EIS and Record of Decision (ROD signed on 19 October 2012) identified the 1960 WCA-3A 3A 9.5 to 10.5 feet, NGVD Regulation Schedule as an interim measure water management criterion for WCA-3A Zone A. This change to Zone A, compared to the previous Interim Operational Plan (IOP) for WCA-3A regulation, was necessary to mitigate for the observed effects, including discharge limitations of the S-12 spillways. The preliminary EN-W analysis of WCA-3A high water levels, which was integrated into the ERTP EIS, also recommended further consideration of additional opportunities to reduce the duration and frequency of Water Conservation Area 3A high water events (ERTP Final EIS, Appendix A-5).

The ERTP analysis of WCA-3A high water events indicated that, based on current system conditions as simulated in the water budget spreadsheet, the IOP infrastructure and operational configuration of WCA-3A would result in a predicted increase in the Standard Project Flood (SPF) stage for WCA-3A of between 1.3 and 1.4 feet compared to the WCA-3A design assumptions (1960 General Design Memorandum (GDM), C&SF Project for Flood Control and Other Purposes, Part I, Supplement 33). Predicted SPF stages are increased from 12.40 to 13.76 feet NGVD and from 13.90 to 15.20 feet NGVD for the S-12 headwater stage and the WCA-3A three-gage average stage, respectively. The ERTP analysis also illustrated, through the use of current USGS rating curves for the S-12 spillways, that the peak SPF stage is increased over the original design due to a reduction in outlet capacity from WCA-3A through the S-12s. This significant change to the original design assumptions, with the additional diminished extent of emergent vegetation within WCA-3A, led the USACE to identify WCA-3A high water stages as a potential cause for concern. Due to the simplistic nature (i.e., volumetric and not hydraulic routing) of the ERTP (Phase 1) analysis, the level of flood protection afforded by WCA-3A was not completely addressed during the initial assessment under ERTP; additional analyses, as identified for inclusion under a subsequent detailed study phase (termed Phase 2 in the ERTP Final EIS), are required to investigate and specify the level of protection afforded by the WCA-3A water management regime and levee configuration.

The preliminary ERTP analysis was limited to a simplified hydrology and hydraulics assessment, while the ERTP-recommended Phase 2 analysis was envisioned to include a more robust hydrology and hydraulics assessment and additional engineering analysis of the structural and geotechnical design aspects for WCA-3A. The recommended Phase 2, which concurrent with CEPP development has remained in the initial scoping and funding phase, is projected to include development of hydrologic/hydraulic models; SPF hydraulic routings for each of the WCAs, to address system changes that have occurred since the original C&SF design; detailed evaluation and risk assessment by hydrology and hydraulic, geotechnical, and structural design engineering disciplines; and quantification of flood protection levels. Upon completion of the SPF routing, additional engineering analysis, incorporating current USACE guidelines for risk analysis requirements will be performed to analyze levee stability and safety issues. This assessment is expected to identify proposed water management operating criteria and potential infrastructure modifications to address identified concerns. Due to the high level of effort and projected time required, USACE recognized that the results from the Phase 2 WCA-3A flood routing hydraulic analysis would not be available for CEPP consideration. Results from Phase 2 will be incorporated into future phases of ERTP, potential future CEPP implementation, or other future regional operational planning efforts, as appropriate.

Although the preliminary ERTP analysis did not provide a quantifiable risk assessment, the hydrologic insights gained from the analysis made it prudent for the USACE to recommend the lowering Zone A of the WCA-3A Regulation Schedule as an interim risk reduction measure, based upon the following considerations:

(1) USGS rating curves and historical discharge data demonstrate limitations in the outlet capacity of the S-12 structures, as compared to the original structure design capacities;
(2) Design storm analysis and SPF event flood routings have not been incorporated into the assessment of WCA-3A Regulation Schedule changes implemented under the Experimental Program, ISOP, and IOP. These analyses are planned for inclusion under the Phase 2 analysis;
(3) The Phase 1 (ERTP) analysis predicted an increase to the WCA-3A high water stages for the SPF design event of 1.3-1.4 feet, compared to the original WCA-3A design assumptions. There is also a recognition that the L-29 Levee/ Tamiami Trail crest elevations (design grade 14.0 feet NGVD) were originally established in conjunction with 1960 WCA-3A regulation Schedule;
(4) Compared to the original WCA-3A design assumptions, the diminished extent of emergent vegetation within WCA-3A may increase the potential effects of wind and wave set-up against the levees;
(5) Zone A lowering is a prudent risk reduction measure that could be implemented expediently with ERTP, which appears to be effective at reducing the peak stage of smaller, more frequent events than the SPF (maximum historical stages correspond to ~50% of SPF).
(6) The lowered Zone A would re-establish consistency with the 1960 regulation schedule, utilized for the original WCA-3A design.
(7) The lowered Zone A will be better able to meet the depths recommended within the FWS Multi-Species Transition Strategy (MSTS) (at critical time periods), providing direct benefits to snail kites, apple snails, wood storks and other wading birds, and tree islands.

Zone A is the top zone of the WCA-3A Regulation Schedule and, when water levels are within Zone A, releases from WCA-3A are to be made up to maximum practicable levels given operational constraints. The ERTP lowering of Zone A represents a return to pre-Experimental Program stage levels for Zone A. The previous IOP Regulation Schedule for WCA-3A included a seasonally varying stage of between 10.75 to 10 feet NGVD in Zone A, while the ERTP schedule has a seasonally varying stage of between 10.5 to 9.5 feet NGVD in Zone A. Flow will overtop the structure gates of the S-12s when the gates are closed, if the S-12 headwater stage exceeds 11.0 feet NGVD; with the gates fully open, the gate clearance elevation is 13.4 feet NGVD. For additional reference, the crest elevation of U.S. Highway 41 (Tamiami Trail) in the reach between S-12A and S-12D is approximately 14.8-14.95 feet NGVD, with slightly lower crest elevations (14.3 feet NGVD) along this portion of the L-29 Levee (Section 2) located further east at S-333.

The information on which the USACE relied on to require the ERTP WCA-3A Zone A as an interim risk reduction measure for WCA-3A high water levels has not changed prior to CEPP formulation, and no new information is currently available compared to the July 2010 assessment included as Appendix A-5 of the ERTP Final EIS. Throughout CEPP formulation, the SAJ Water Resources

Engineering Branch (EN-W) advocated that CEPP formulation efforts attempt to maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP Future Without Project (FWO) condition, which includes ERTP, given recognition of the WCA-3A high water concerns identified with ERTP; prior to CEPP formulation, the January 2012 CEPP Risk Register explicitly recognized that the ERTP constraint precluded raising of the top of the WCA-3A Regulation Schedule, while simultaneously recognizing that substantial benefits were still expected and that goals to further lower stages in WCA-3A were consistent with the constraint. EN-W also indicated that it would continue to rely on the WCA-3A three-gage average stages for assessment of WCA-3A high water frequency, durations, and peak stages, consistent with the original WCA-3A design assumptions and the ERTP assessment; increased weight would not be considered for a single gage, such as 3A-28 (Site 65). It was further noted that if CEPP can provide operational assurances of additional WCA-3A outlet capacity under high water conditions, including adequate consideration of potential WCA-3B seepage management and/or ecological operational limitations, the EN-W may be able to further consider proportional relaxation of the WCA-3A FWO high water duration and frequency targets.

Preliminary CEPP formulation efforts for the Green and Blue Line components, which relied on the iModel, were not able to demonstrate achievement of the FWO frequency of time within Zone A of the ERTP WCA-3A Regulation Schedule, based on the system-wide optimization of ecological targets and consideration of the additional ~220,000 acre-feet (220 kAF) of inflows to WCA-3A available from the Flow Equalization Basin (FEB) and associated water quality treatment (refer to section 3.2.3 of the CEPP PIR main report and Appendix E.1 for additional discussion). Significant increases in WCA-3A regulatory discharge capacity were also not identified through the preliminary iModel screening.

The requirements to maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP Future Without Project (FWO) condition were actively integrated into the formulation efforts to identify the CEPP final array of alternatives, and the assessment of the final array demonstrated levels of performance consistent with this requirement. The EN-W assessment relied on additional post-processing of the RSM-GL model results, as subsequently discussed.

3.2.1.1. WCA-3A High Water Performance Criteria

To establish the WCA-3A high water performance criteria to assist with CEPP formulation and to provide technical recommendations to the CEPP formulation efforts, EN-W developed comparisons between the ERTP Recommended Plan modeling (Alternative 9E1 in the ERTP Final EIS), which was simulated with the SFWMM, and the RSM-GL base conditions representations that were developed for CEPP starting in May 2012. Based on the results of these comparisons, EN-W recommended in July 2012 that CEPP formulation efforts should identify alternative configurations which maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP FWO condition. No significant changes to WCA-3A stage duration curves were observed for subsequent incremental iterations of the SFWMM ECB and FWO base conditions that were generated in August 2012 and December 2012, and, therefore the original EN-W WCA-3A high water performance criteria were retained throughout CEPP formulation efforts.

Following USACE vertical team endorsement of the CEPP modeling strategy in January 2012 (Decision Point 1), it was known that SFWMM modeling simulations would not be conducted for CEPP. Different from ERTP, initial assessments for CEPP formulation screening and alternative evaluations for WCA-3A were based on RSM-GL results, including a 1965-2005 period of simulation (the ERTP SFWMM modeling period of simulation was 1965-2000). The CEPP FWO Baseline modeling assumed ERTP operations, in addition to the following additional assumptions potentially affecting WCA-3A: operation of the SFWMD A-1 FEB to achieve water quality compliance (note: this feature was not modeled for the draft FWO baseline simulations in May 2012 and August 2012, pending additional details from the then-concurrent SFWMD Restoration Strategies modeling efforts and water quality compliance coordination); operation of Compartments B and C (STA-2 and STA-5, respectively); operation of Broward Water Preserve Areas (BCWPA) CERP project; and completion of the Tamiami Trail 1-mile bridge (G-3273 constraint remains in place, and L-29 constraint remains at 7.5 feet NGVD). Since ERTP was modeled with the SFWMM (1965-2000 period of simulation), the ERTP simulation results are not directly comparable to the CEPP modeling.

To provide a meaningful comparison between ERTP SFWMM modeling and the CEPP baselines, the SFWMM and RSM-GL comparisons were limited to the 1965-2000 time period. Stage hydrographs for the WCA-3A three-gage average stage (average of the 3A-3, 3A-4, and 3A-28 monitoring gages; refer to the Figure 7 map) are displayed in Figure 10 for the ERTP SFWMM Recommended Plan (Alternative 9E1), and the final CEPP RSM-GL ECB and FWO baseline simulations; Figure 10 includes a sample time period for 1993-2005, which includes representative extreme wet and dry conditions within the ERTP SFWMM and CEPP RSM-GL simulation periods, with the seasonally-varying ERTP WCA-3A Regulation Schedule Zone A line shown for reference. Compared to the ERTP SFWMM modeling, the CEPP RSM-GL FWO baseline indicates an approximately 0.25 foot lowering in the upper 10 percent of the stage duration curve for the WCA-3A three-gage average stage, as shown in Figure 11 (full stage duration curve) and Figure 12 (upper 25 percent of the stage duration curve). In order to consider potential differences during specific years, the EN-W assessment also considered the annual duration of exceedance of the ERTP WCA-3A Zone A stage levels for the comparison time period (Figure 13). The annual durations were also displayed and assessed as a frequency curve (Figure 14). Given consideration of the across-model comparison, the differences in assumptions between the ERTP SFWMM modeling and the CEPP RSM-GL FWO modeling, and the ERTP engineering-based recommendations to lower the frequency, duration, and peak stage of WCA-3A high water levels, the RSM-GL FWO simulation was recommended by EN-W to serve as an upper bound for WCA-3A high water levels for CEPP formulation. Figures 11 through 14 additionally show the lowered WCA-3A water levels with ERTP through comparison of the CEPP ECB (IOP operations) and the CEPP FWO, as well as the insignificant effects on peak WCA-3A stages and Zone A exceedance with the RSM-GL FWO update for inclusion of the A-1 FEB operations (comparing the May 2012 RSM-GL FWO version to the final December 2012RSM-GL FWO simulation).

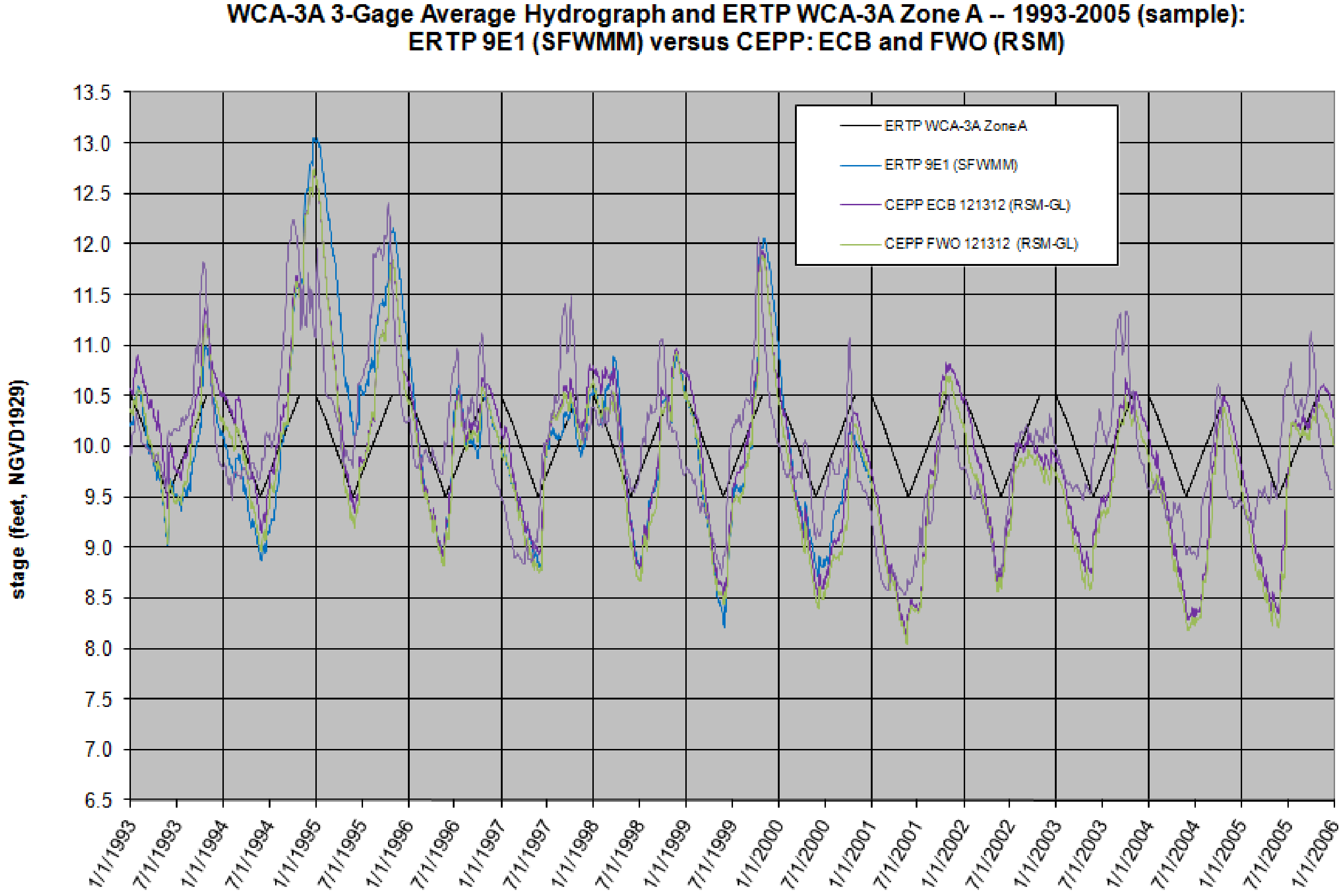


Figure 10: WCA-3A 3-gage average hydrographs for ERTP SFWMM modeling and CEPP RSM-GL baselines

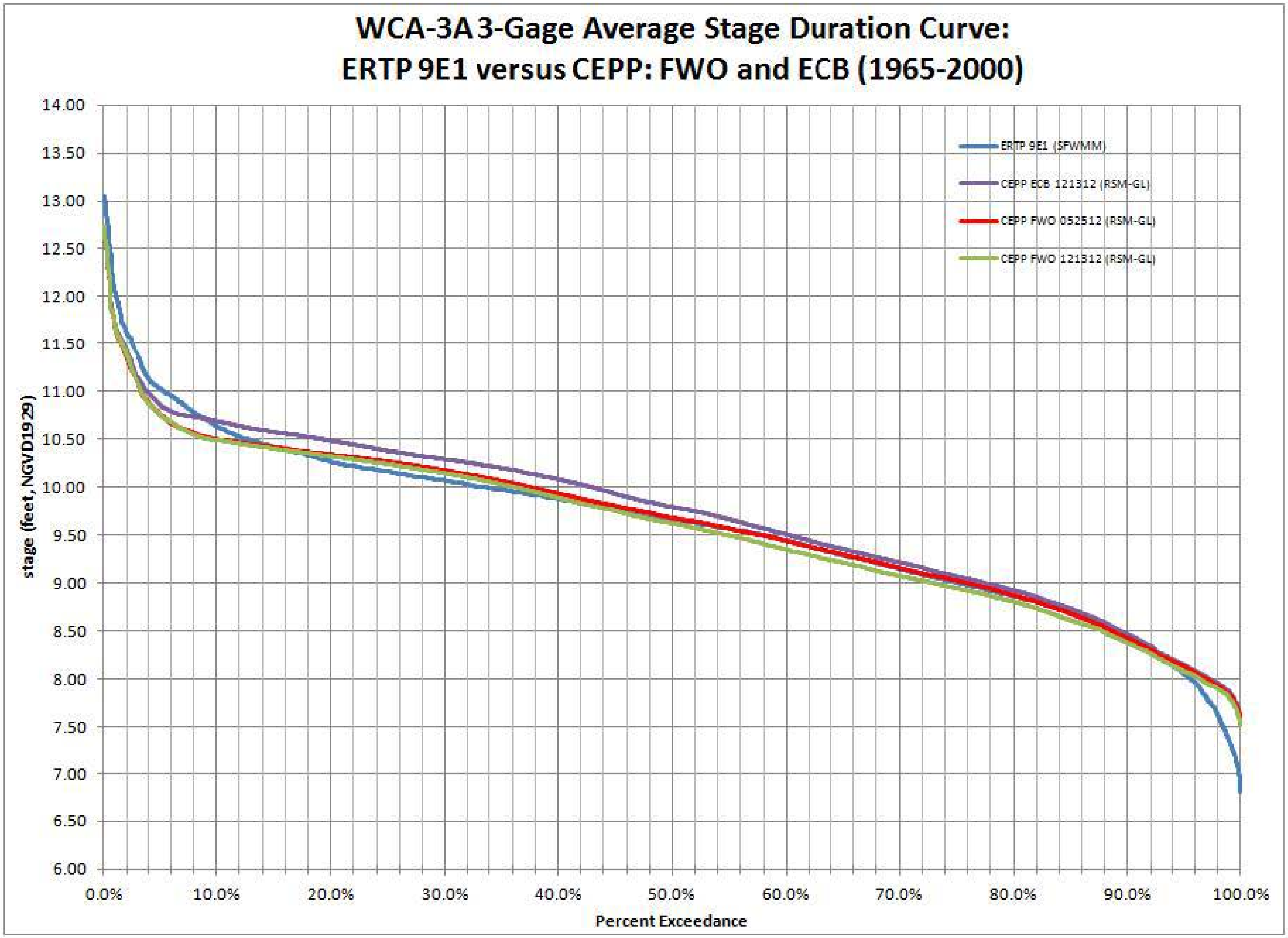


Figure 11: WCA-3A 3-gage average stage duration curves for ERTP SFWMM modeling and CEPP RSM-GL baselines

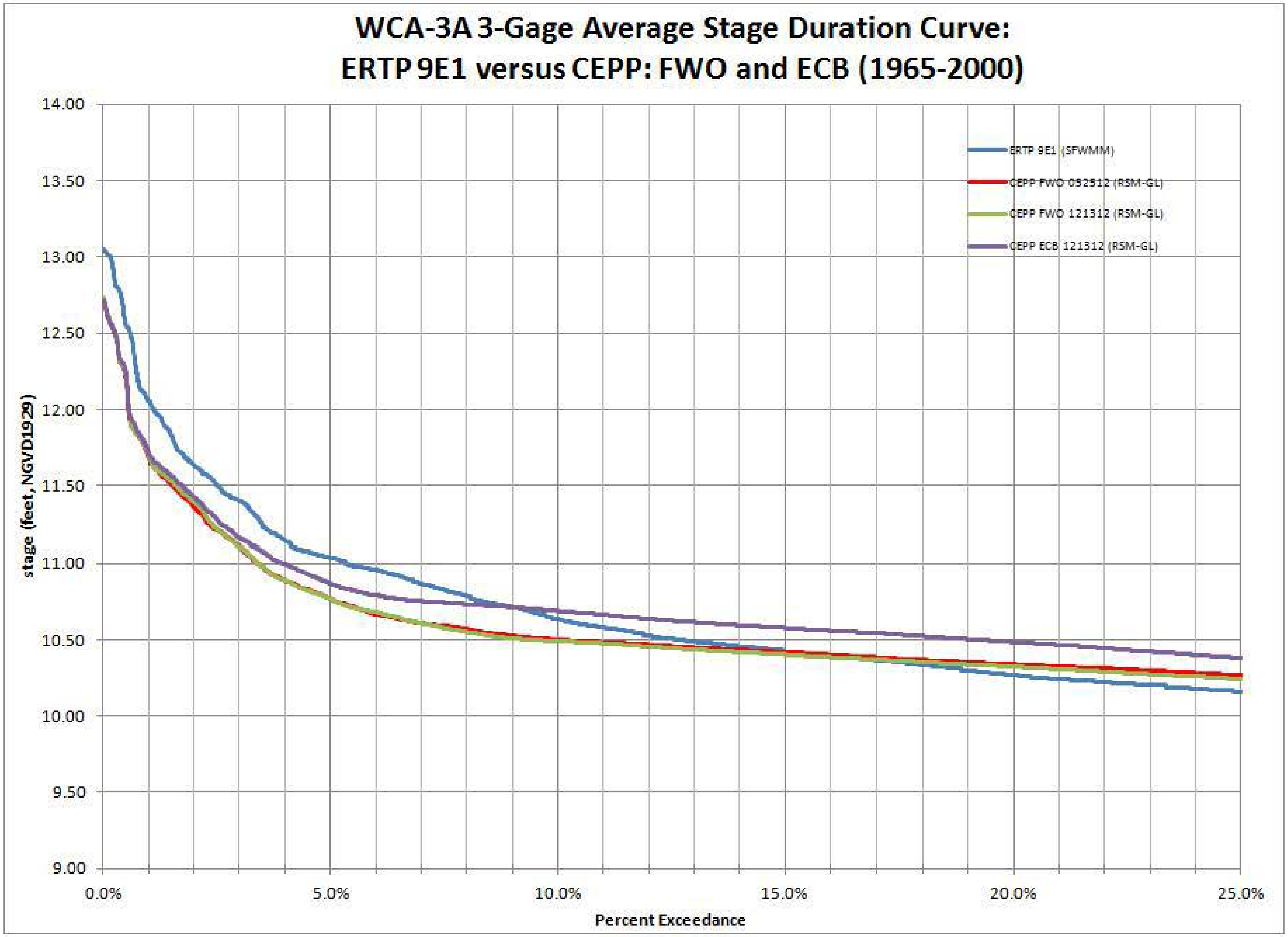


Figure 12: WCA-3A 3-gage average stage duration curves for ERTP SFWMM modeling and CEPP RSM-GL baselines (Upper 25%)

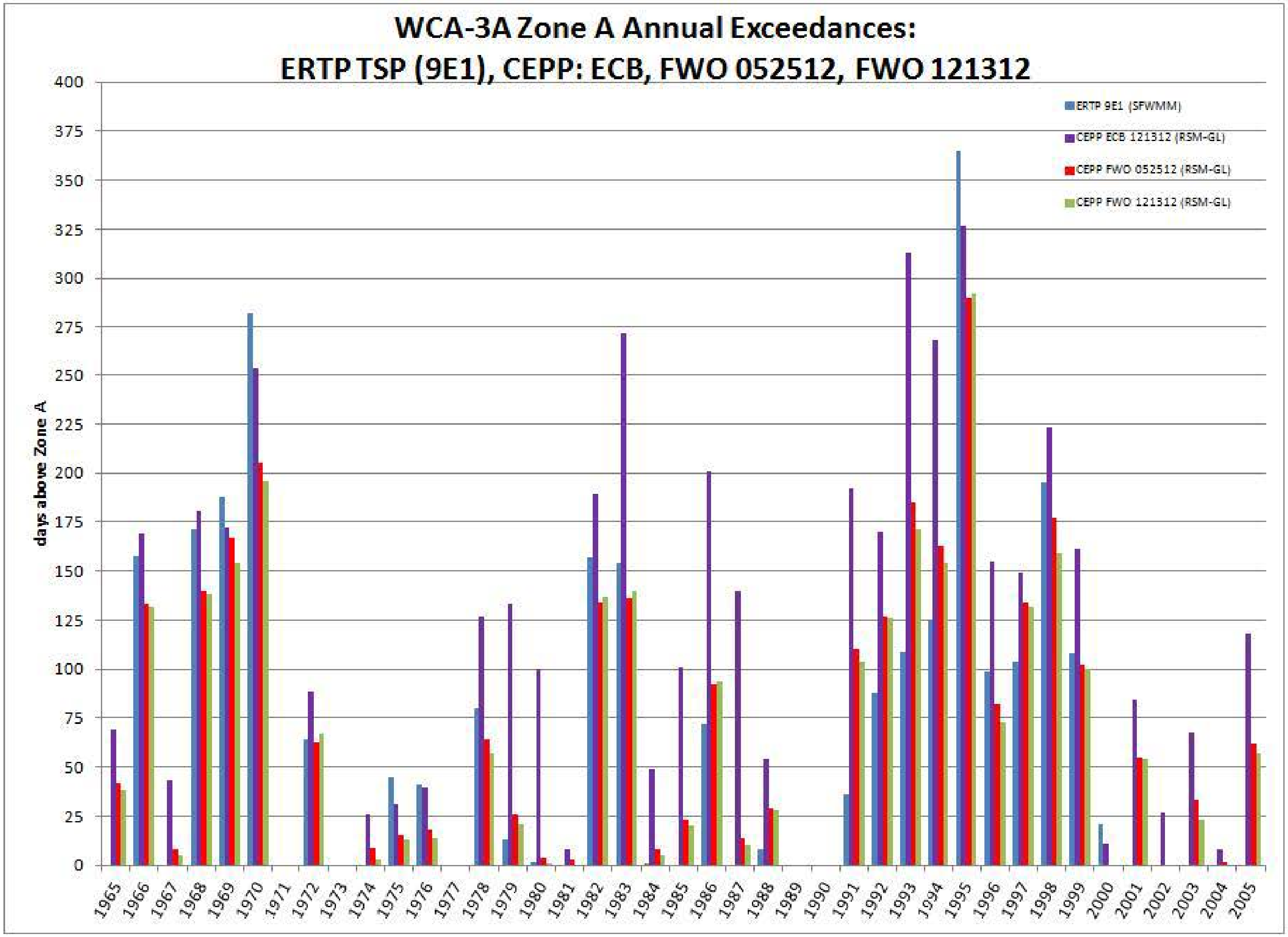


Figure 13: WCA-3A 3-gage average annual Zone A exceedance for ERTP SFWMM modeling and CEPP RSM-GL baselines

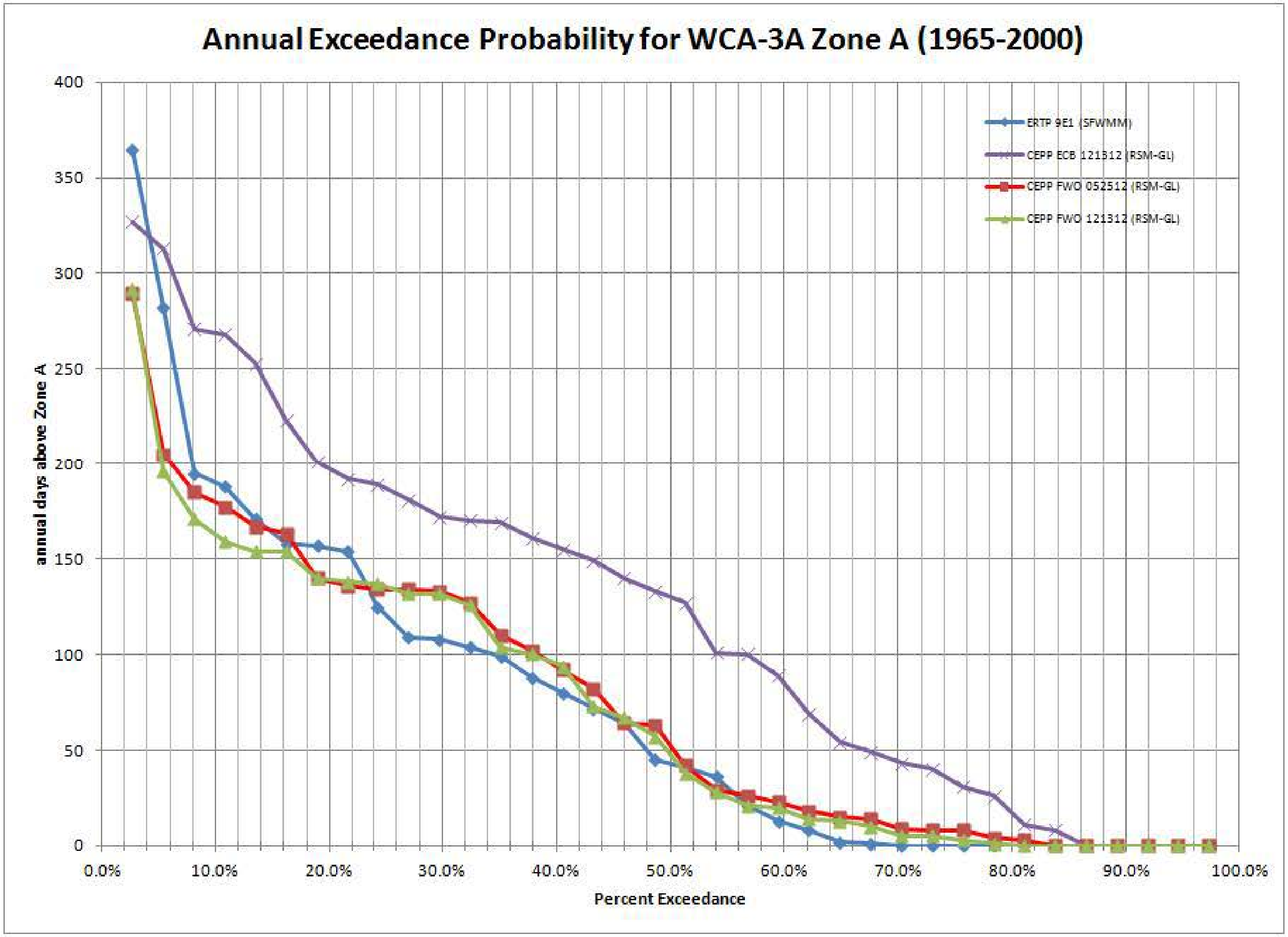


Figure 14: WCA-3A 3-gage average duration curve for annual Zone A exceedance for ERTP SFWMM modeling and CEPP RSM-GL baseline

3.2.1.2. WCA-3A High Water Performance Evaluation

The requirements to maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP FWO condition were actively integrated into the formulation efforts to identify the CEPP final array of alternatives, and the CEPP modeling team considered these requirements as a constraint during the modeling of the final array of alternatives.

Applying the EN-W recommendation to utilize the CEPP FWO as the upper bounds for WCA-3A high water performance, the performance of the CEPP final array of alternatives for WCA-3A high water conditions was initially assessed in January 2013 for Alternatives 1 through 4 and updated in June-July 2013 for Alternatives 4R and 4R2. The CEPP FWO and the CEPP final array of alternatives were each simulated with the RSM-GL, with the complete 1965-2005 period of simulation. Example stage hydrographs for the WCA-3A three-gage average stage are displayed in Figure 15 (Alternatives 1 through 4) and Figure 16 (Alternatives 4, 4R, and 4R2) for the CEPP FWO and the CEPP final array of alternatives for a sample time period for 1993-2005, with the seasonally-varying ERTP WCA-3A Regulation Schedule Zone A line shown for reference. Compared to the CEPP FWO (final December 2012 release), the CEPP alternatives are lowered by approximately 0.1-0.3 feet in the upper 10 percent of the stage duration curve for the WCA-3A three-gage average stage, as shown in Figures 17-18 (full stage duration curve) and Figures 19-20 (upper 25 percent of the stage duration curve). In order to consider potential differences during specific years, the EN-W assessment also considered the annual duration of exceedance of the ERTP WCA-3A Zone A stage levels for the complete period of simulation (Figures 21-22). The annual durations were also displayed and assessed as a frequency curve (Figure 23-24). The total number of days above Zone A is summarized as follows for the CEPP FWO and CEPP alternatives (with percent of total period of simulation, 14975 days, in parentheses): CEPP FWO – 2718 days (18.15%); Alternative 1 – 3206 days (21.41%); Alternative 2 – 3034 days (20.26%); Alternative 3 – 3285 days (21.94%); Alternative 4 – 3227 days (21.55%); Alternative 4R – 3307 days (22.08%); and Alternative 4R2 – 3323 days (22.19%).

The EN-W performance assessment for the final array of alternatives further reviewed the WCA-3A stage hydrographs for individual years in which the number of days above Zone A increased by more than 20 percent between the CEPP FWO and any of the CEPP alternatives, as shown highlighted in Table 5 and Table 6. Annual hydrographs are also provided for each of the twelve years which triggered this further detailed assessment (Figures 25 through 38): 1969, 1980, 1983-1985, 1993-1996, 1999, 2003, and 2005.

Annual stage hydrograph statistical distribution plots were developed to assist with the general characterization of differences in intra-annual stage variability, to facilitate comparisons between the CEPP ECB baseline condition, the CEPP FWO baseline condition, and the CEPP final array of alternatives (refer to Figures 39 through 46). For the 41-year period of simulation, the graphics illustrate the maximum and minimum stage, 90th and 10th percentile stages, 75th and 25th percentile stages, median stage, and mean stage at a daily time step. The graphics also include the ERTP WCA-3A Regulation Schedule Zone A reference line, the FWS MSTS recommended seasonal range for January 1 and May 1-31, and the average ground surface elevation (GSE) for the WCA-3A 3-gage average at 8.34 feet NGVD (3A-3 GSE 9.08 feet NGVD; 3A-4 GSE 8.49 feet NGVD; 3A-28 GSE 7.44 feet NGVD). Compared to the CEPP FWO, the following general trends are noted for the CEPP alternatives (which all perform similarly for

WCA-3A stages): increased stages through the dry season, particularly January through May (most evident for the 75th and 90th percentiles); increased stages at the end of the dry season in May (most evident for 10th through 90th percentiles); increased stages at the beginning of the wet season in June-July (evident under all conditions); increased stages through the wet season and start of the dry season during average to dry years (evident for minimum to median stages); reduced stages at the end of the wet season in September-October during wet years (90th percentile and maximum stage); and reduced stages at the beginning of the dry season in November and December during wet years (90th percentile and maximum stages). These graphics illustrate that the increased durations within Zone A with the CEPP alternatives, as compared to the CEPP FWO, are the combined result of higher stages at the end of the dry season (along the Zone A recession) and higher antecedent stages at the beginning of the wet season (June 1) with the resulting effects of early wet season rainfall events. Peak stages and durations of Zone A exceedance at the end of the wet season, when WCA-3A design limitations are most critical due to the maximum stages, do not increase and, in many instances, are significantly reduced compared to the FWO condition. This conclusion is consistent with detailed review of the annual hydrographs presented in Figure 25 through 38. To facilitate direct comparisons between Alternative 4 and the operational refinements to the NER Plan Alternative 4M (Alternatives 4R and 4R2), WCA-3A 3-gage average statistical distribution plots were specifically generated for the mean daily stage hydrograph (Figure 47), 25th percentile daily stage hydrograph (Figure 48), 75th percentile daily stage hydrograph (Figure 49), maximum daily stage hydrograph (Figure 50), and minimum daily stage hydrograph (Figure 51) for the complete RSM-GL simulation period-of-record (POR) 1965-2005. Figure 52 provides a mean daily stage difference hydrograph, which compares the intra-annual variability of the TSP Alternative 4R2 with the ECB, FWO, Alternative 4, and Alternative 4R. Comparison hydrographs for Alternative 4 and the operational refinements to the NER Plan are also displayed for selected wet (1995, 1995, 1999) and dry (1989, 2001) years in Figures 53 through 57.

The detailed EN-W assessment of the frequency, duration, and peak stages of high water levels within WCA-3A concluded: (1) WCA-3A peak stages are lowered (these stages are most critical for WCA-3A design limitations); (2) the frequency and durations of Zone A exceedance are increased; (3) the increased frequency and durations occur during periods of the year when WCA-3A water levels are below peak critical levels; (4) CEPP infrastructure modifications (increased WCA-3A outlet capacity) and operations demonstrate that increased WCA-3A stages at the end of the dry season and start of the wet season can be effectively managed to avoid exacerbating high water conditions at the end of the wet season when Zone A levels off at 10.5 feet NGVD; and (5) CEPP infrastructure and operations utilized to achieve these performance levels need to be codified in the CEPP Project Operating Manual (POM). The requirements to maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP FWO were, therefore, successfully achieved based on EN-W assessment of the overall performance of the CEPP final array, including the TSP.

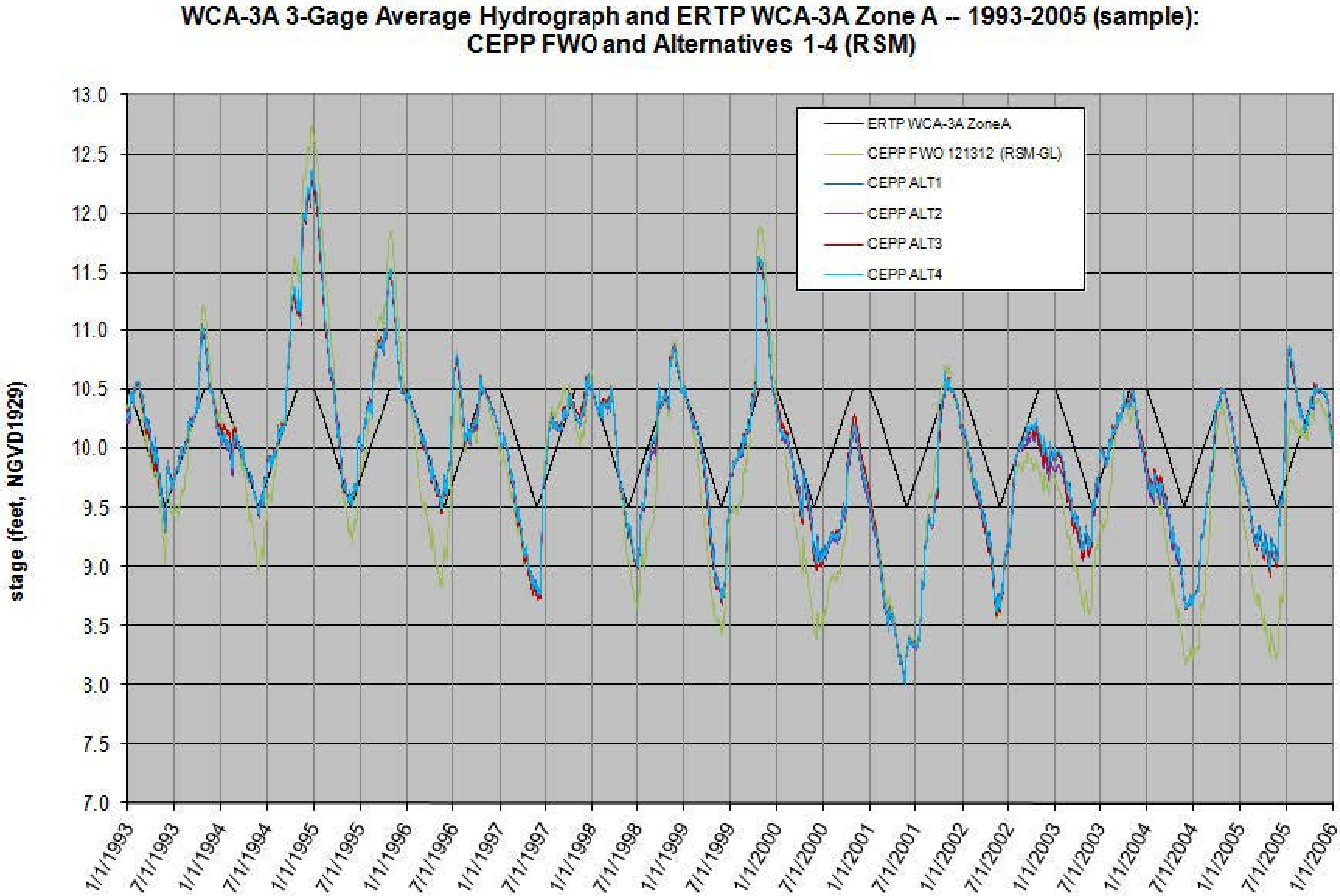


Figure 15: WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4

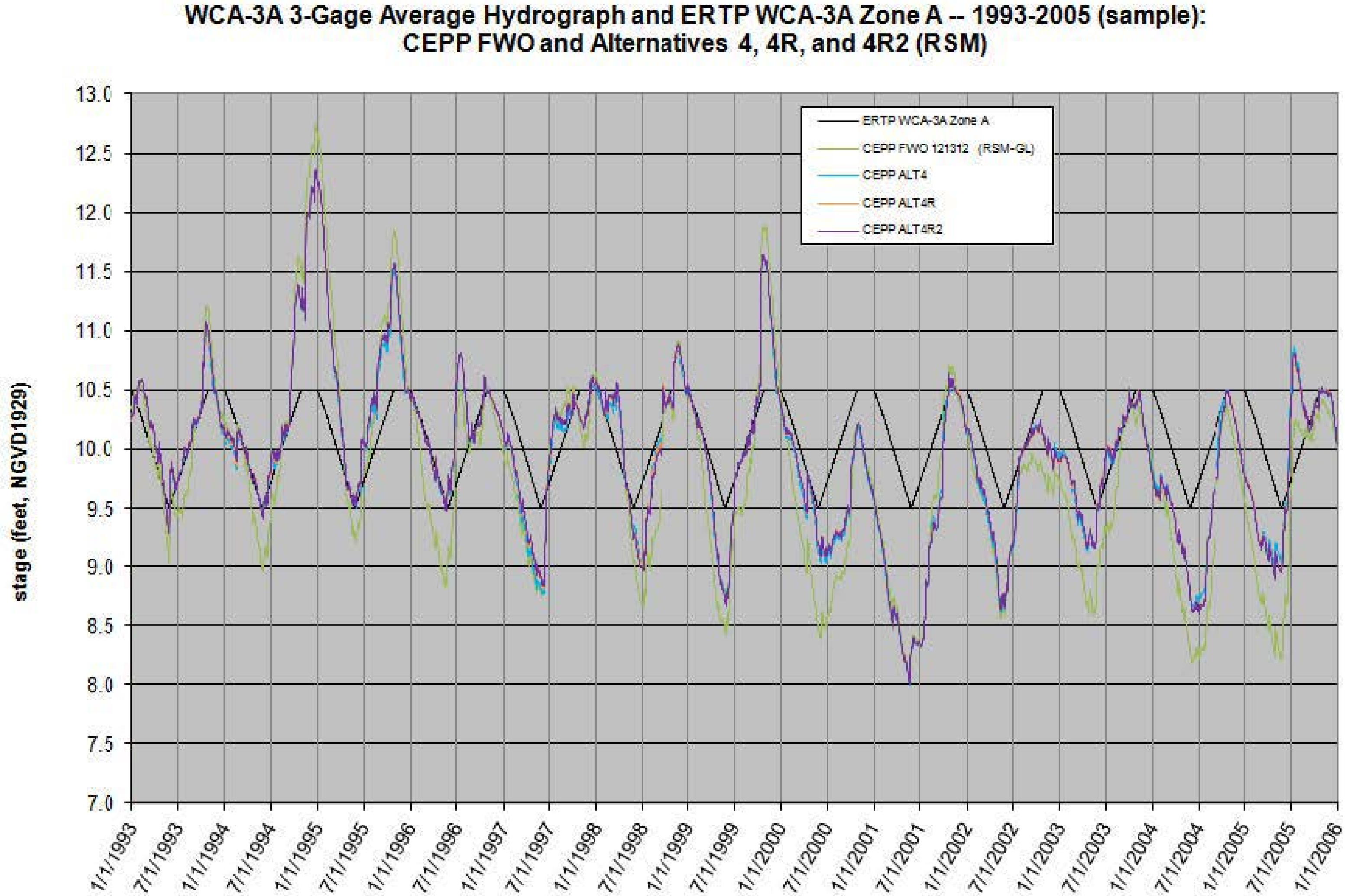


Figure 16: WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

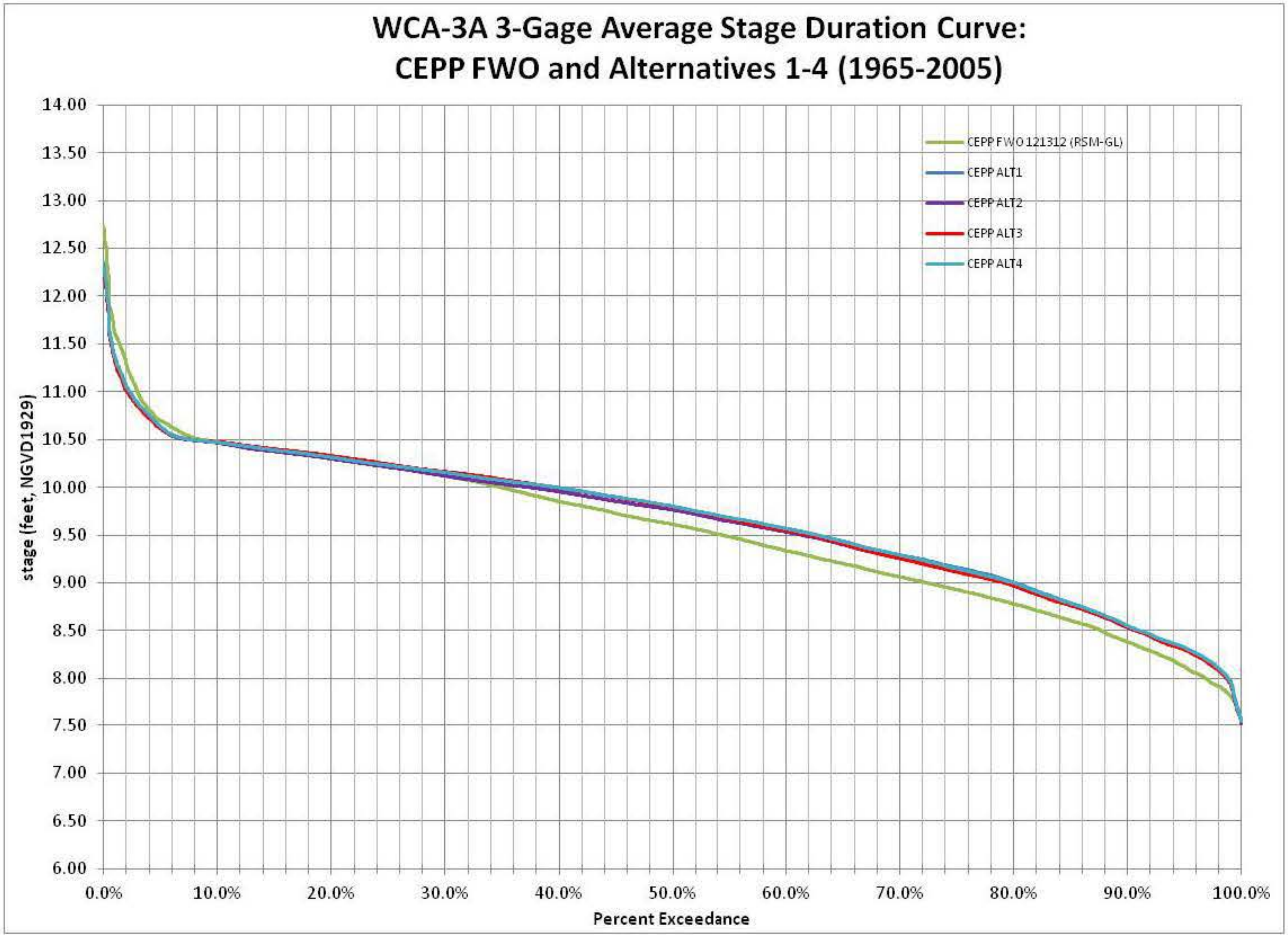


Figure 17: WCA-3A 3-gage average stage duration curve for CEPP FWO and CEPP Alternatives 1 through 4

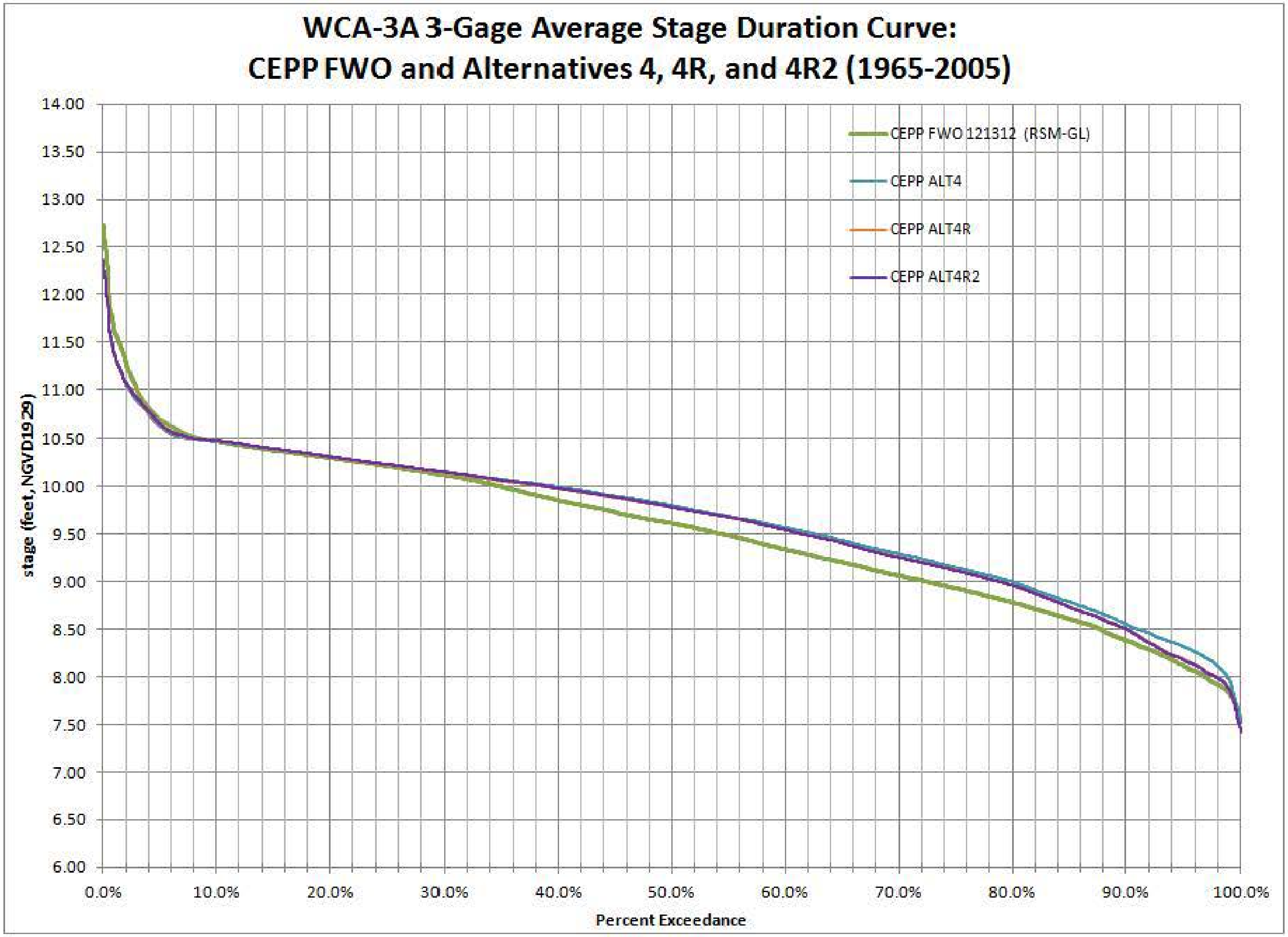


Figure 18: WCA-3A 3-gage average stage duration curve for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

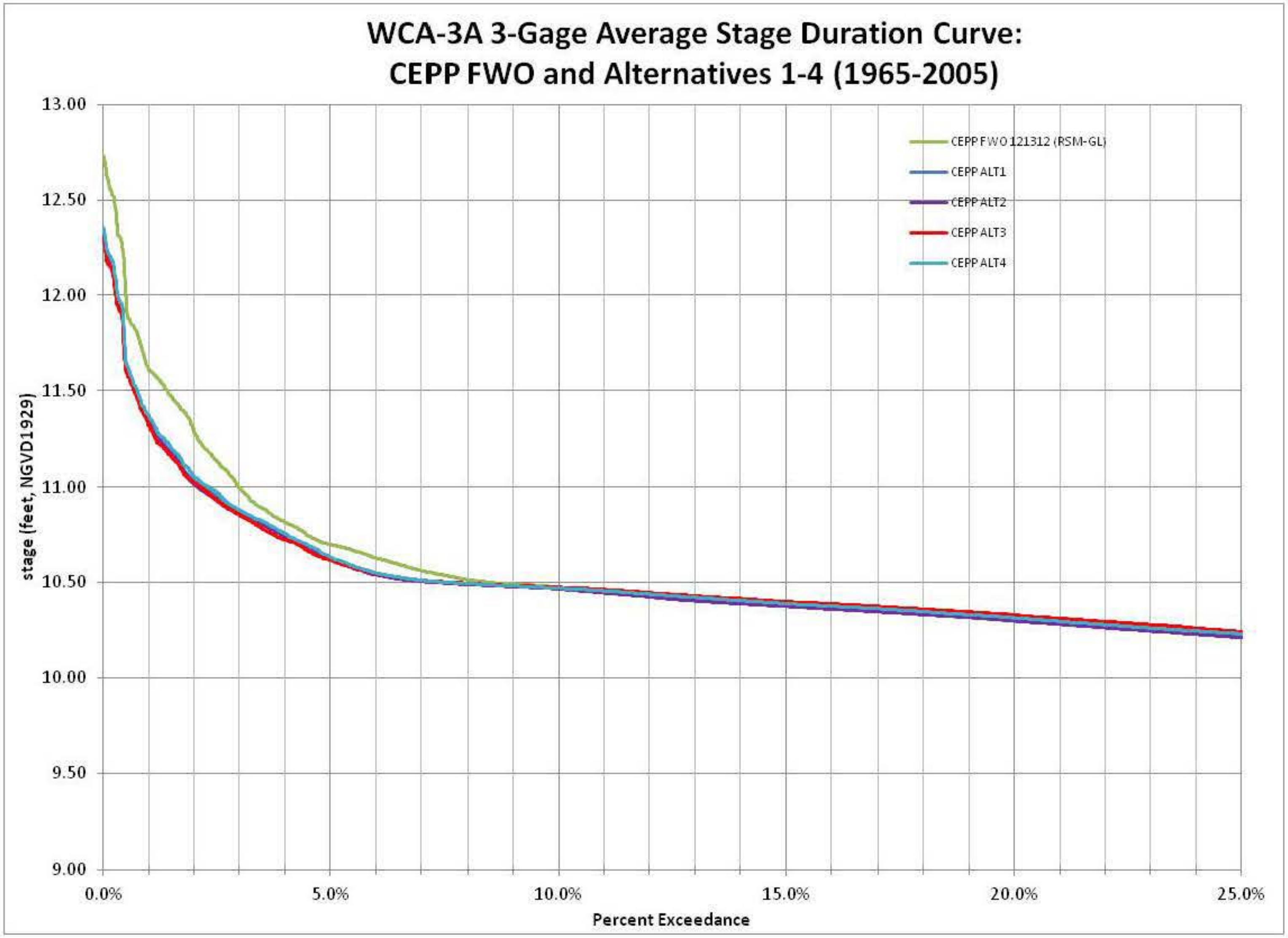


Figure 19: WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4 (Upper 25%)

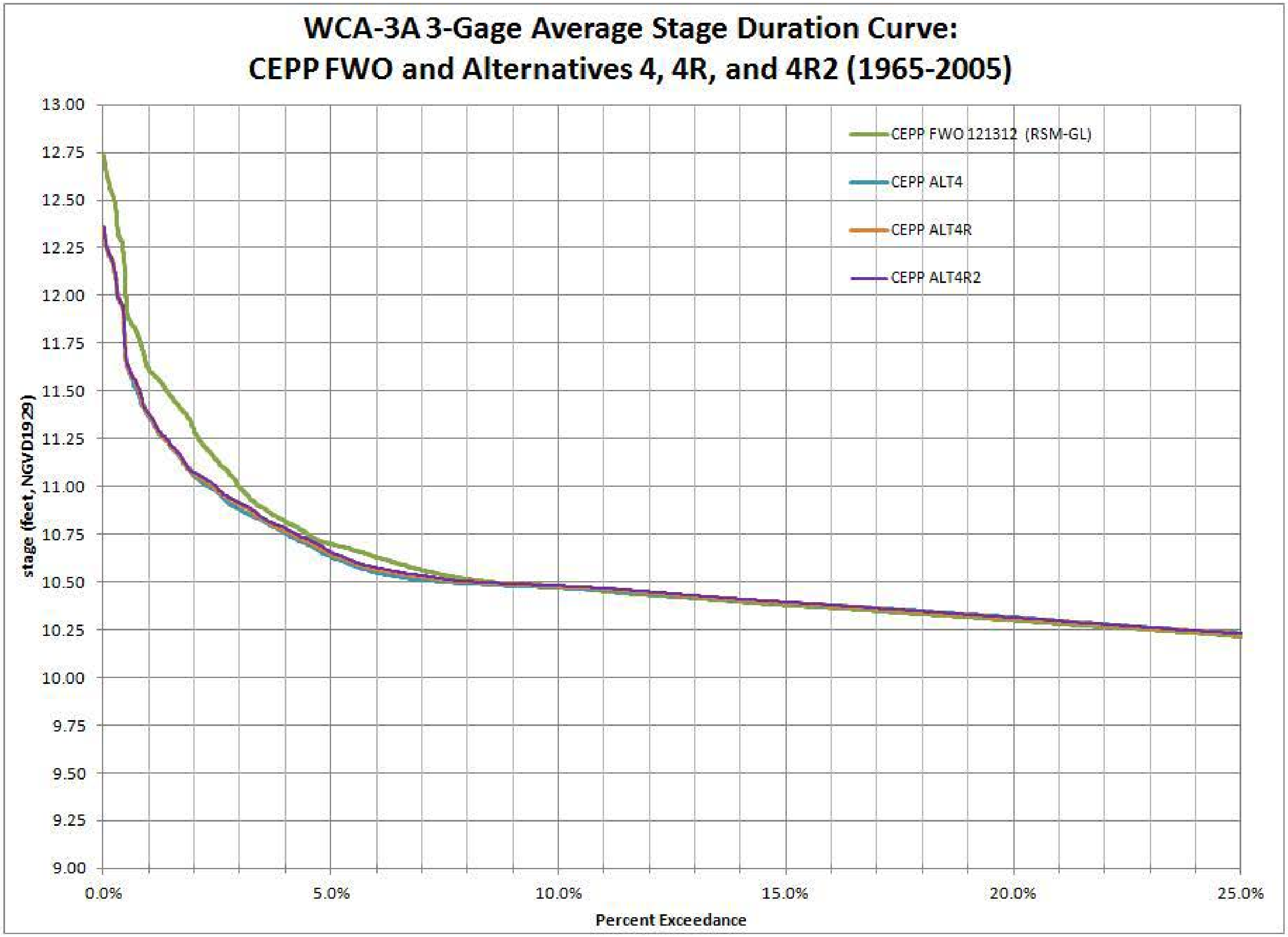


Figure 20: WCA-3A 3-gage average stage duration curve for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2 (upper 25%)

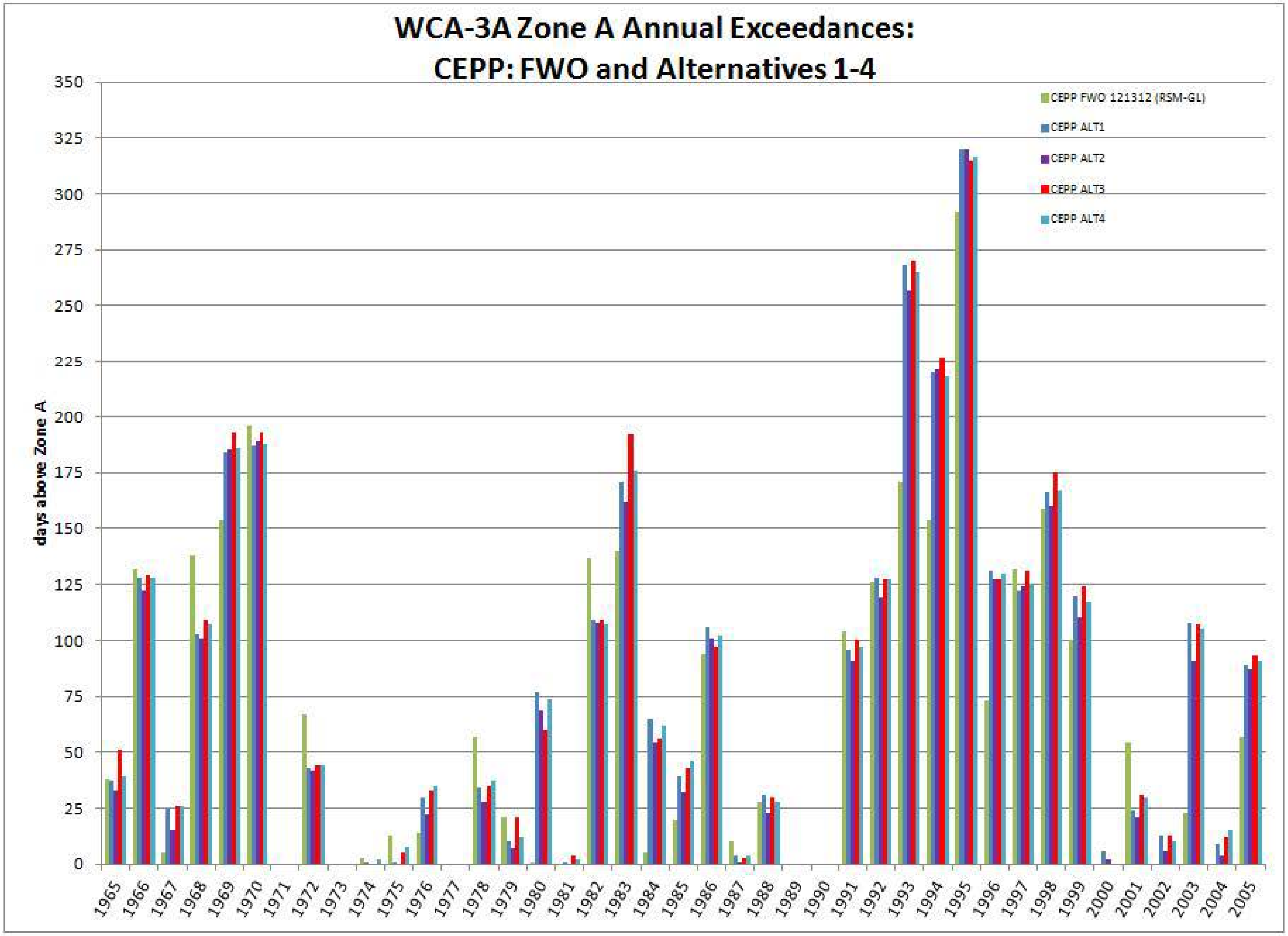


Figure 21: WCA-3A 3-gage average annual Zone A exceedance for CEPP FWO and CEPP Alternatives 1 through 4

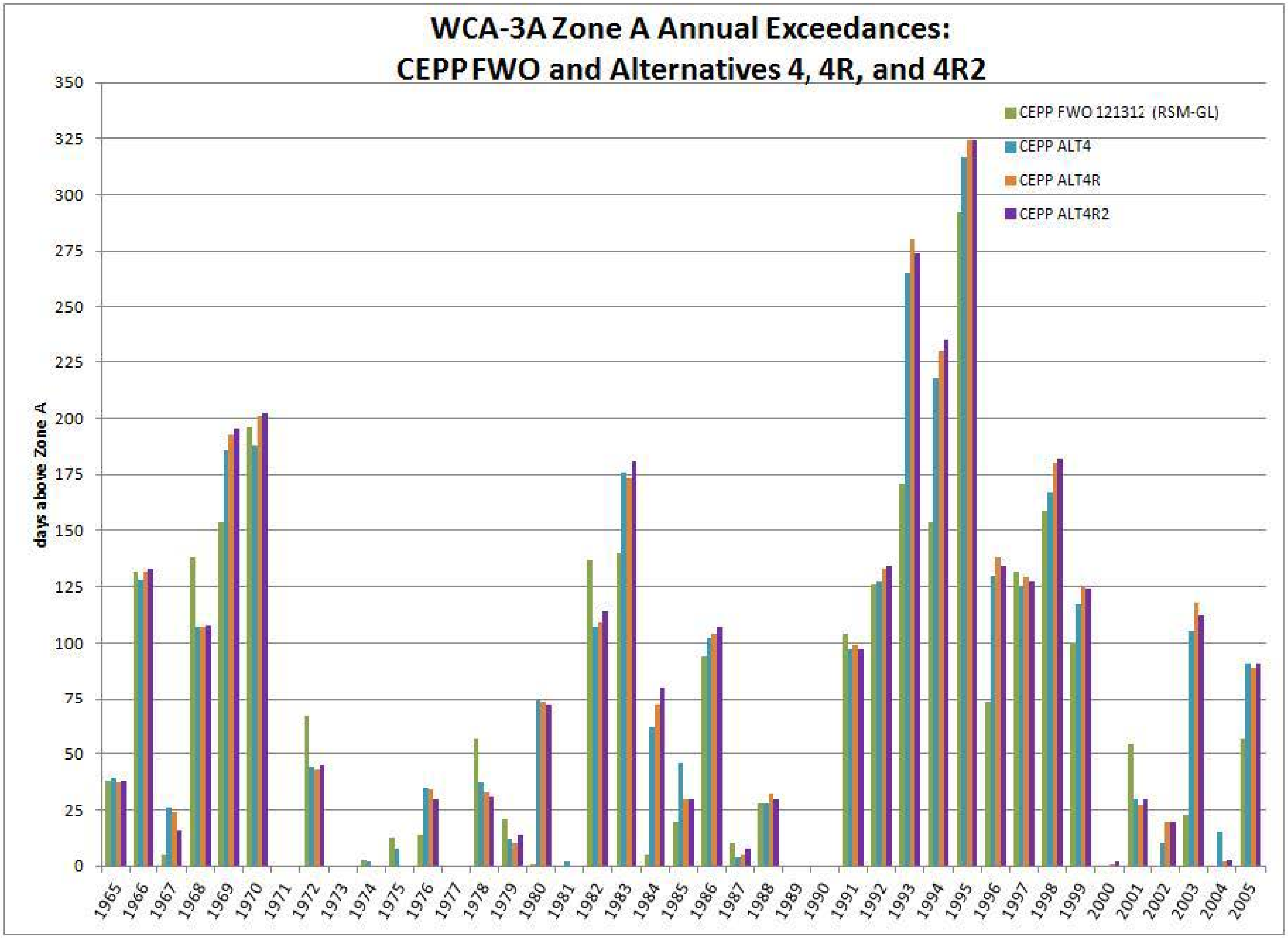


Figure 22: WCA-3A 3-gage average annual Zone A exceedance for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

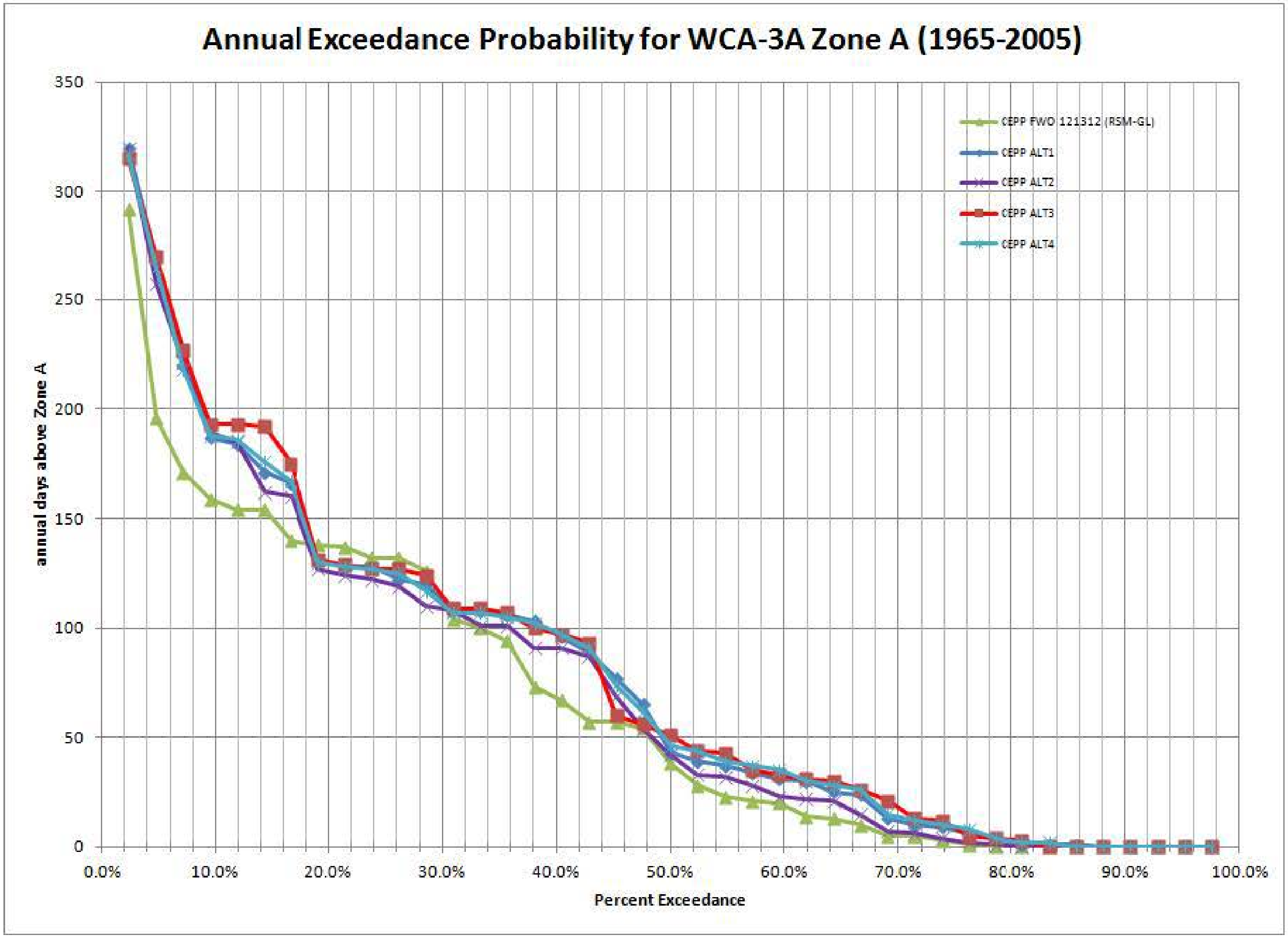


Figure 23: WCA-3A 3-gage average duration curve for annual Zone A exceedance for FWO and CEPP Alternatives 1 through 4

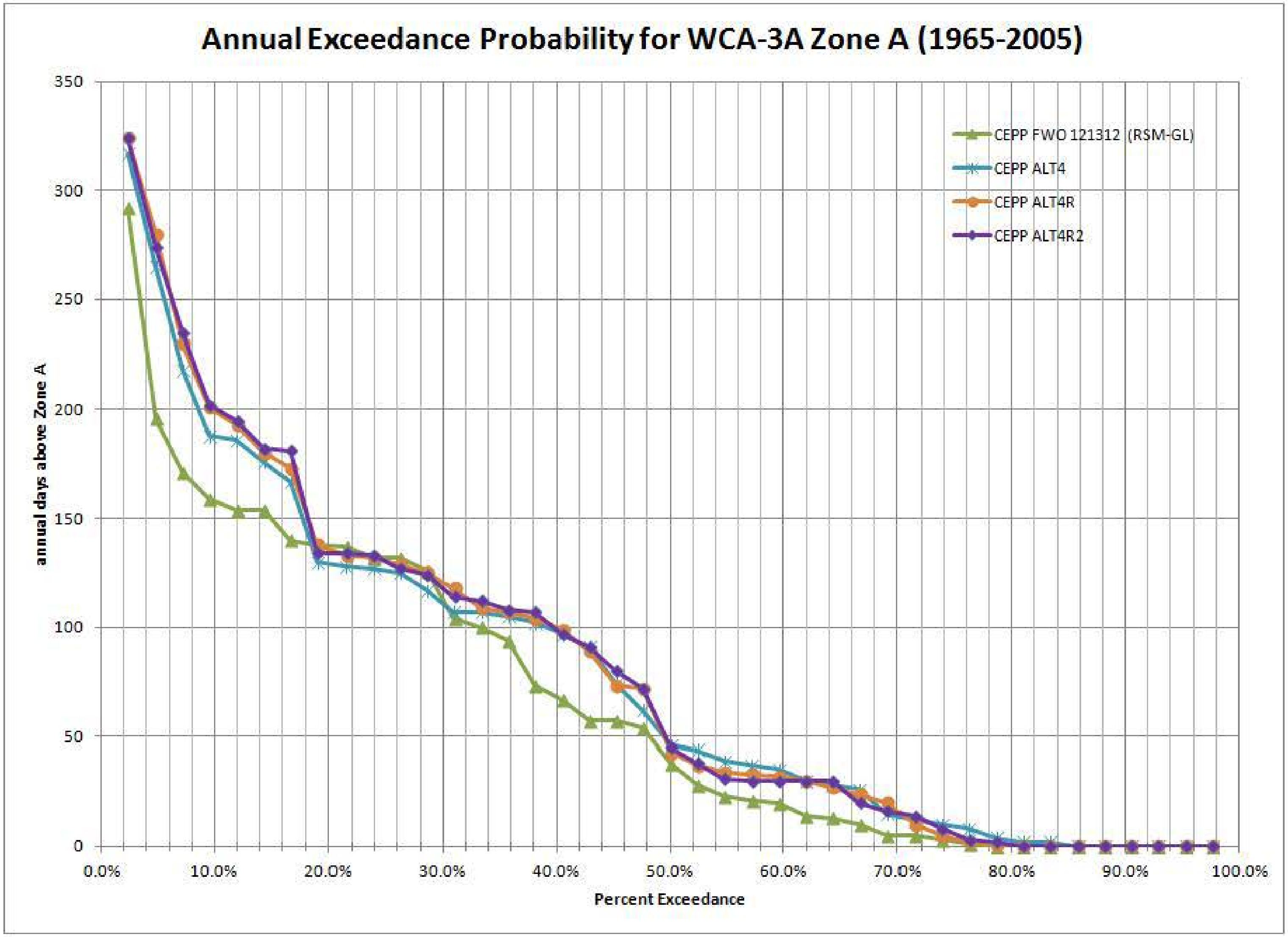


Figure 24: WCA-3A 3-gage average duration curve for annual Zone A exceedance for FWO and CEPP Alternatives 4, 4R, and 4R2

Table 5: Annual Zone A exceedance days (WCA-3A 3-gage average) for FWO and CEPP Alternatives 1 through 4

	Summary Table: WCA-3A Zone A Annual Exceedance Duration				
Year	**CEPP FWO 121312**	**CEPP ALT1**	**CEPP ALT2**	**CEPP ALT3**	**CEPP ALT4**
1965	38	37	33	51	39
1966	132	128	122	129	128
1967	5	25	15	26	26
1968	138	103	101	109	107
1969	154	184	185	193	186
1970	196	187	189	193	188
1971	0	0	0	0	0
1972	67	43	42	44	44
1973	0	0	0	0	0
1974	3	1	0	0	2
1975	13	1	0	5	8
1976	14	30	22	33	35
1977	0	0	0	0	0
1978	57	34	28	35	37
1979	21	10	7	21	12
1980	1	77	69	60	74
1981	0	1	0	4	2
1982	137	109	108	109	107
1983	140	171	162	192	176
1984	5	65	54	56	62
1985	20	39	32	43	46
1986	94	106	101	97	102
1987	10	4	1	3	4
1988	28	31	23	30	28
1989	0	0	0	0	0
1990	0	0	0	0	0
1991	104	96	91	100	97
1992	126	128	119	127	127
1993	171	268	257	270	265
1994	154	220	221	227	218
1995	292	320	320	315	317
1996	73	131	127	127	130
1997	132	122	124	131	125
1998	159	166	160	175	167
1999	100	120	110	124	117
2000	0	6	2	0	0
2001	54	24	21	31	30
2002	0	13	6	13	10
2003	23	108	91	107	105
2004	0	9	4	12	15
2005	57	89	87	93	91
	Summary Table: WCA-3A Zone A Annual Exceedance Duration				
	CEPP FWO 121312	**CEPP ALT1**	**CEPP ALT2**	**CEPP ALT3**	**CEPP ALT4**
total (1965-2005 POR; 14975 days)	2718	3206	3034	3285	3227
total (percent of POR)	**18.15**	**21.41**	**20.26**	**21.94**	**21.55**
percent increase vs FWO	--	17.95	11.63	20.86	18.73

Table 6: Annual Zone A exceedance days (WCA-3A 3-gage average) for FWO and CEPP Alternatives 4, 4R, and 4R2

	Summary Table: WCA-3A Zone A Annual Exceedance Duration (days)			
Year	**CEPP FWO 121312**	**CEPP ALT4**	**CEPP ALT4R**	**CEPP ALT4R2**
1965	38	39	37	38
1966	132	128	132	133
1967	5	26	24	16
1968	138	107	107	108
1969	154	186	193	195
1970	196	188	201	202
1971	0	0	0	0
1972	67	44	43	45
1973	0	0	0	0
1974	3	2	0	0
1975	13	8	0	0
1976	14	35	34	30
1977	0	0	0	0
1978	57	37	33	31
1979	21	12	10	14
1980	1	74	73	72
1981	0	2	0	0
1982	137	107	109	114
1983	140	176	173	181
1984	5	62	72	80
1985	20	46	30	30
1986	94	102	104	107
1987	10	4	5	8
1988	28	28	32	30
1989	0	0	0	0
1990	0	0	0	0
1991	104	97	99	97
1992	126	127	133	134
1993	171	265	280	274
1994	154	218	230	235
1995	292	317	324	324
1996	73	130	138	134
1997	132	125	129	127
1998	159	167	180	182
1999	100	117	125	124
2000	0	0	1	2
2001	54	30	27	30
2002	0	10	20	20
2003	23	105	118	112
2004	0	15	2	3
2005	57	91	89	91
	Summary Table: WCA-3A Zone A Annual Exceedance Duration			
	CEPP FWO 121312	**CEPP ALT4**	**CEPP ALT4R**	**CEPP ALT4R2**
total (1965-2005 POR; 14975 days)	2718	3227	3307	3323
total (percent of POR)	**18.15**	**21.55**	**22.08**	**22.19**
percent increase vs FWO	--	18.73	21.67	22.26

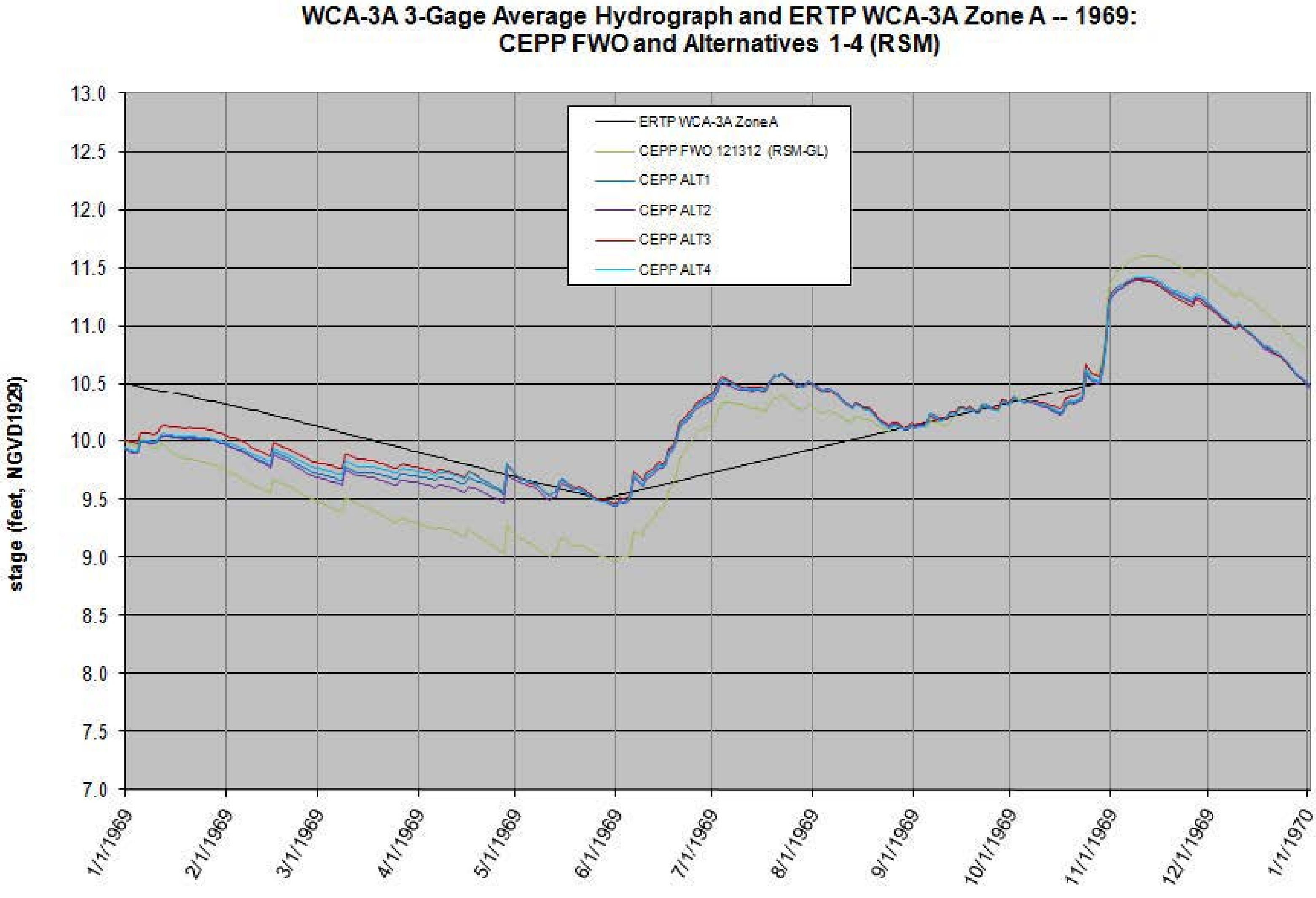


Figure 25: 1969 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4

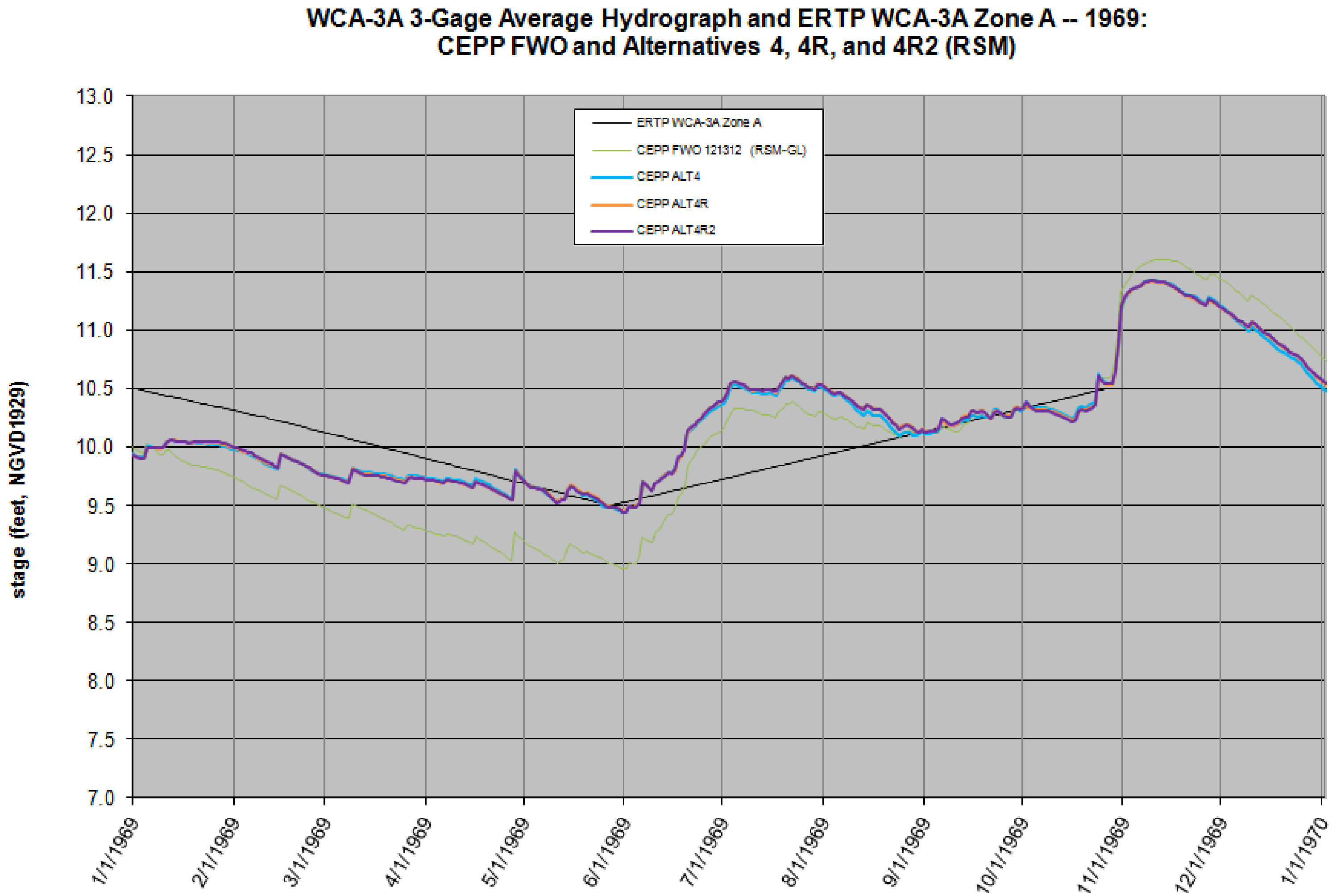


Figure 26: 1969 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

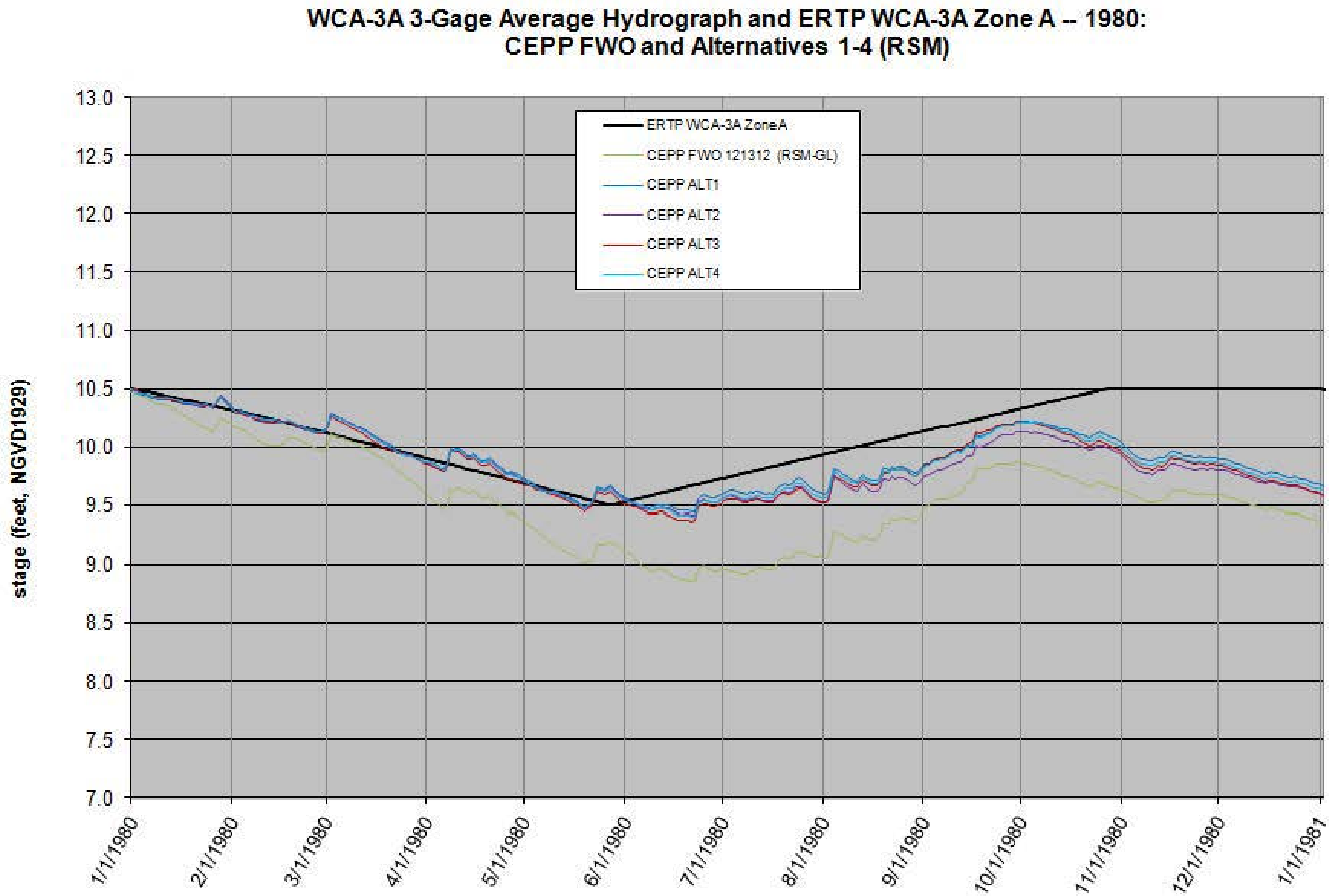


Figure 27: 1980 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4

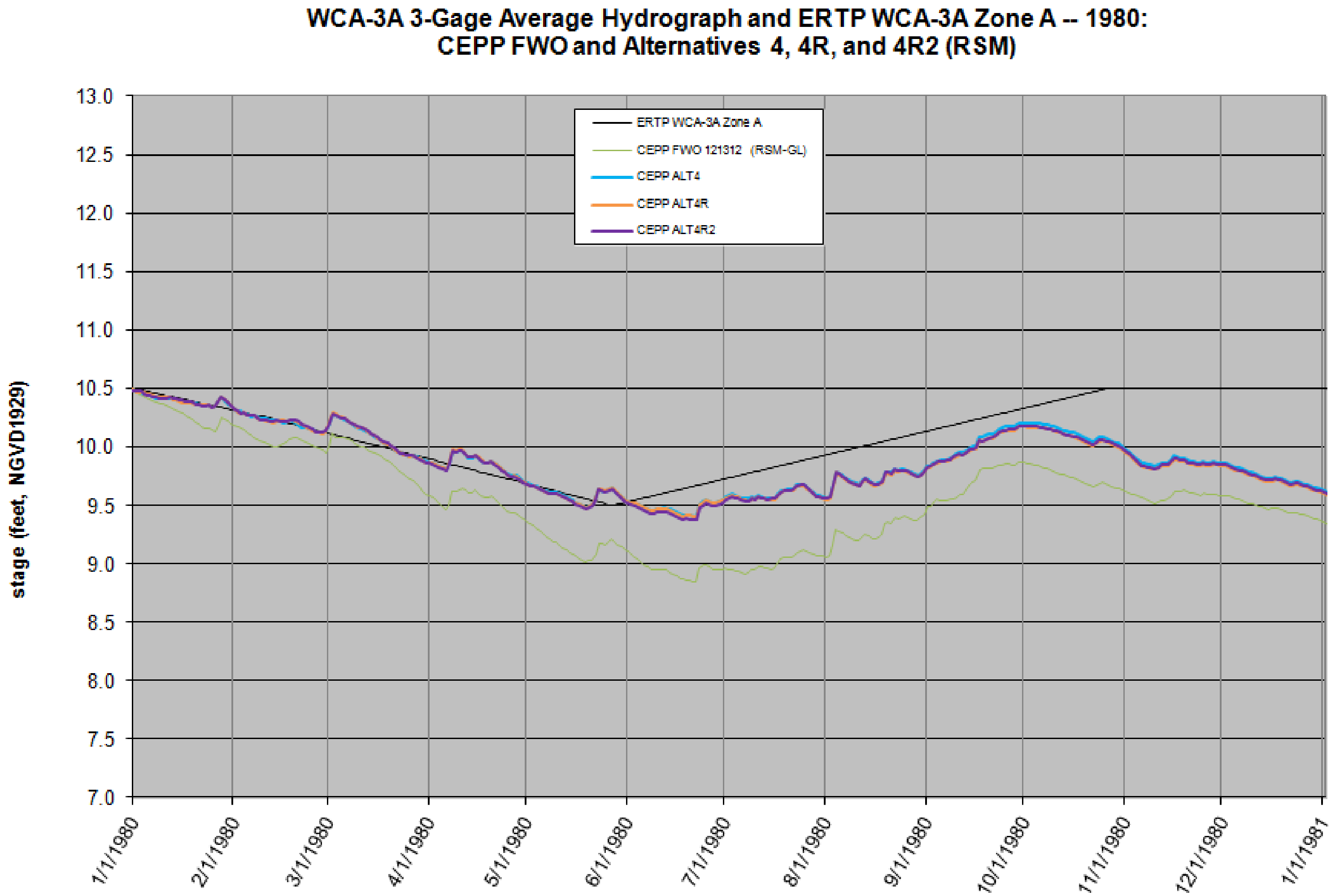


Figure 28: 1980 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

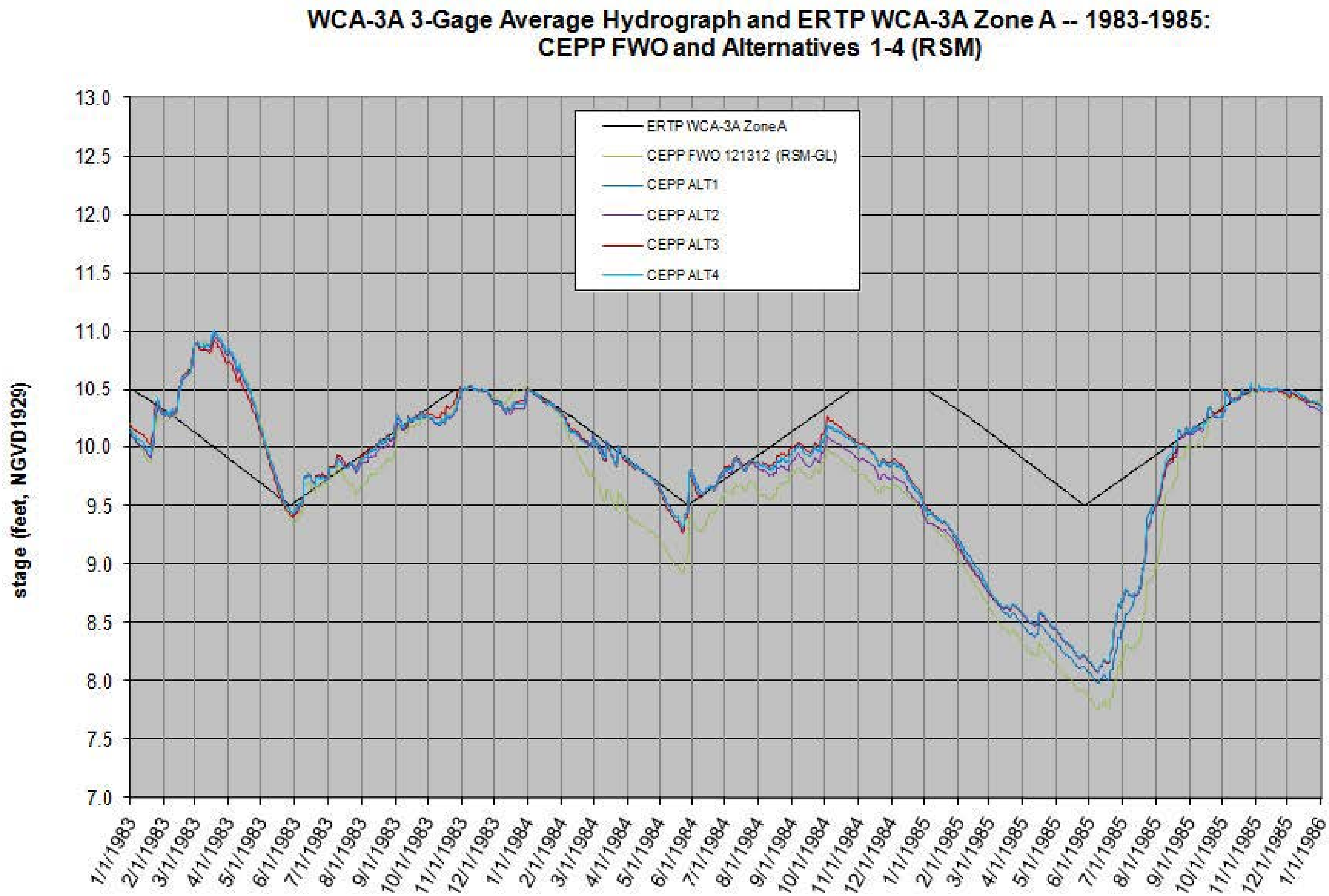


Figure 29: 1983-1985 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4

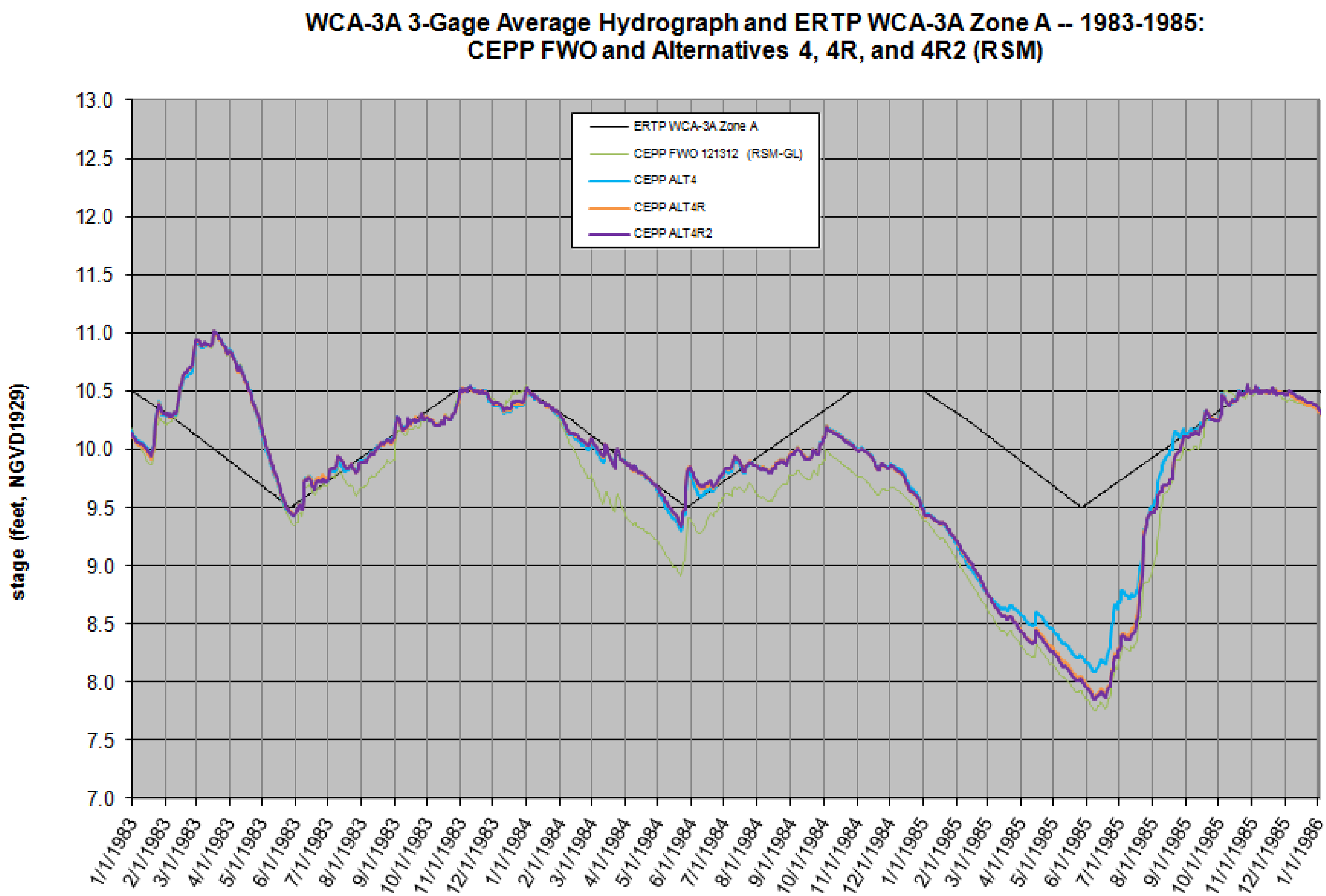


Figure 30: 1983-1985 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

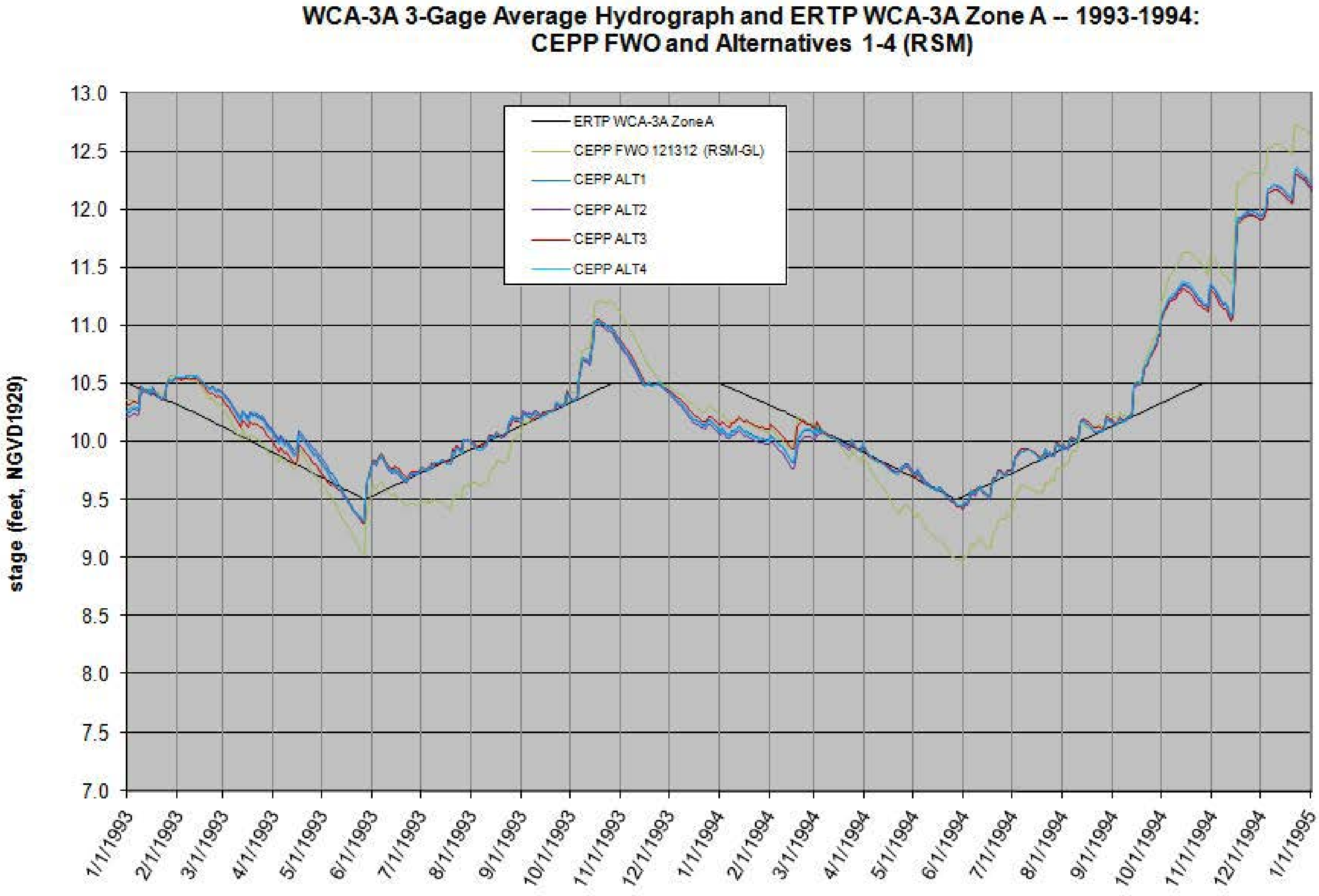


Figure 31: 1993-1994 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4

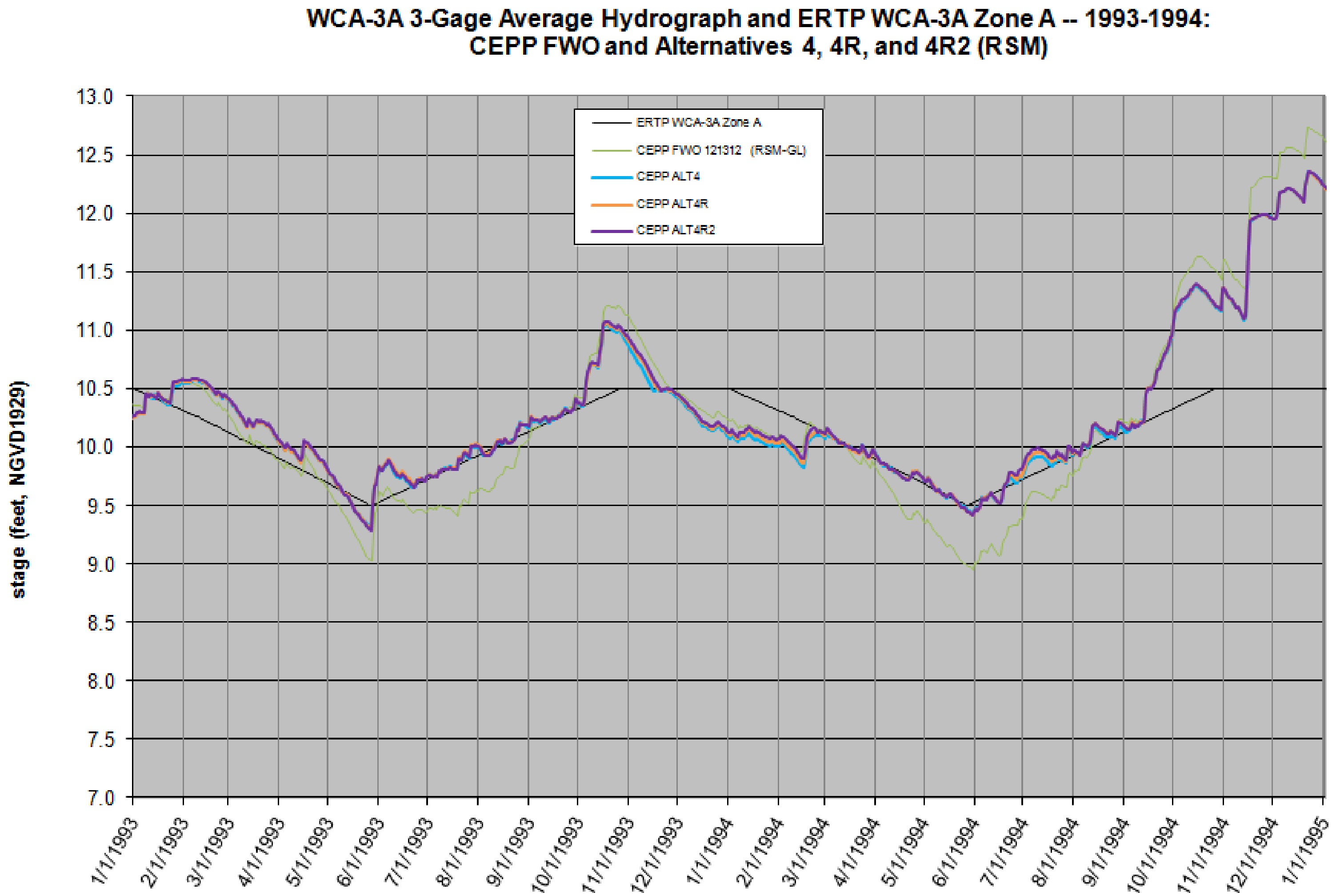


Figure 32: 1993-1994 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

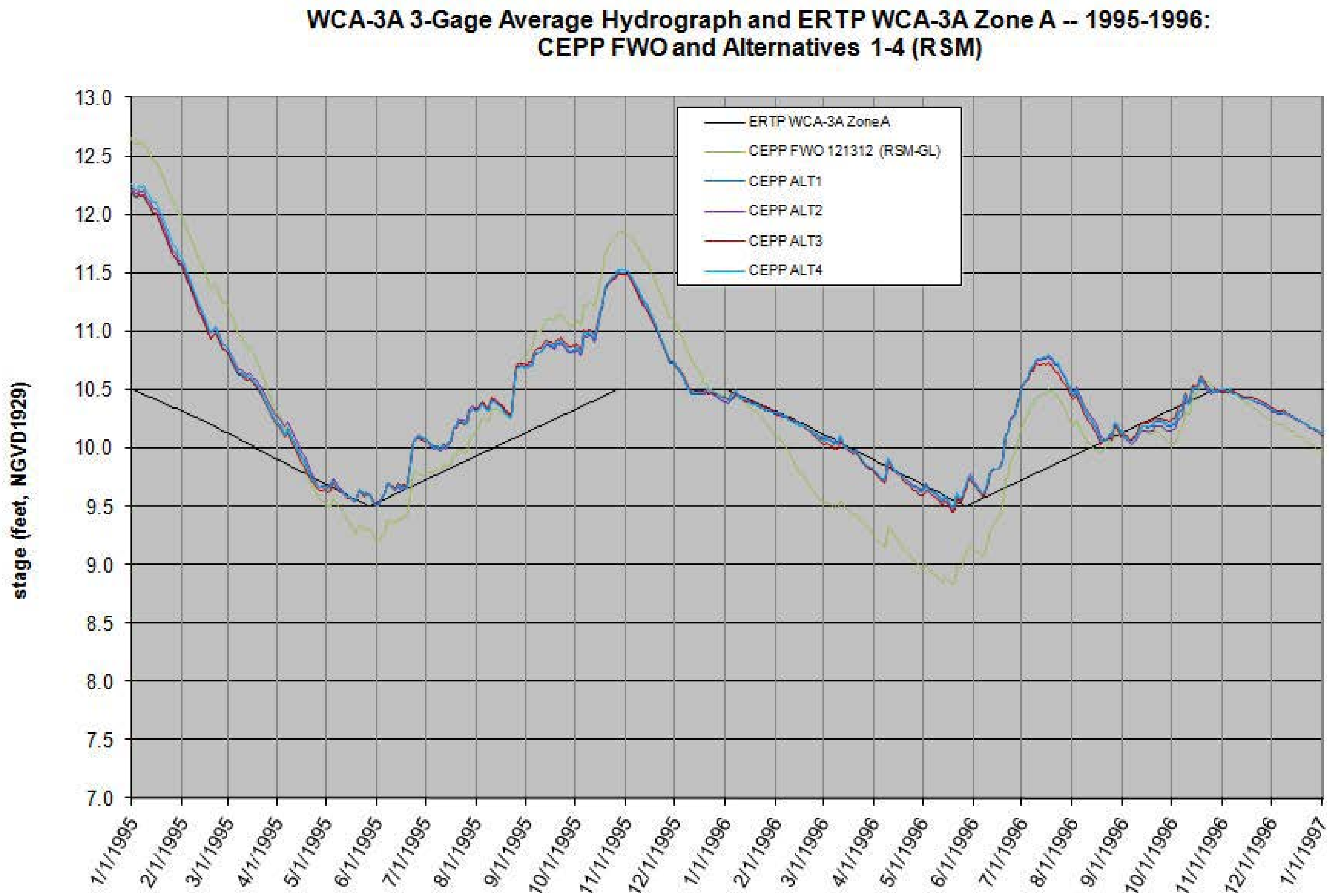


Figure 33: 1995-1996 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4

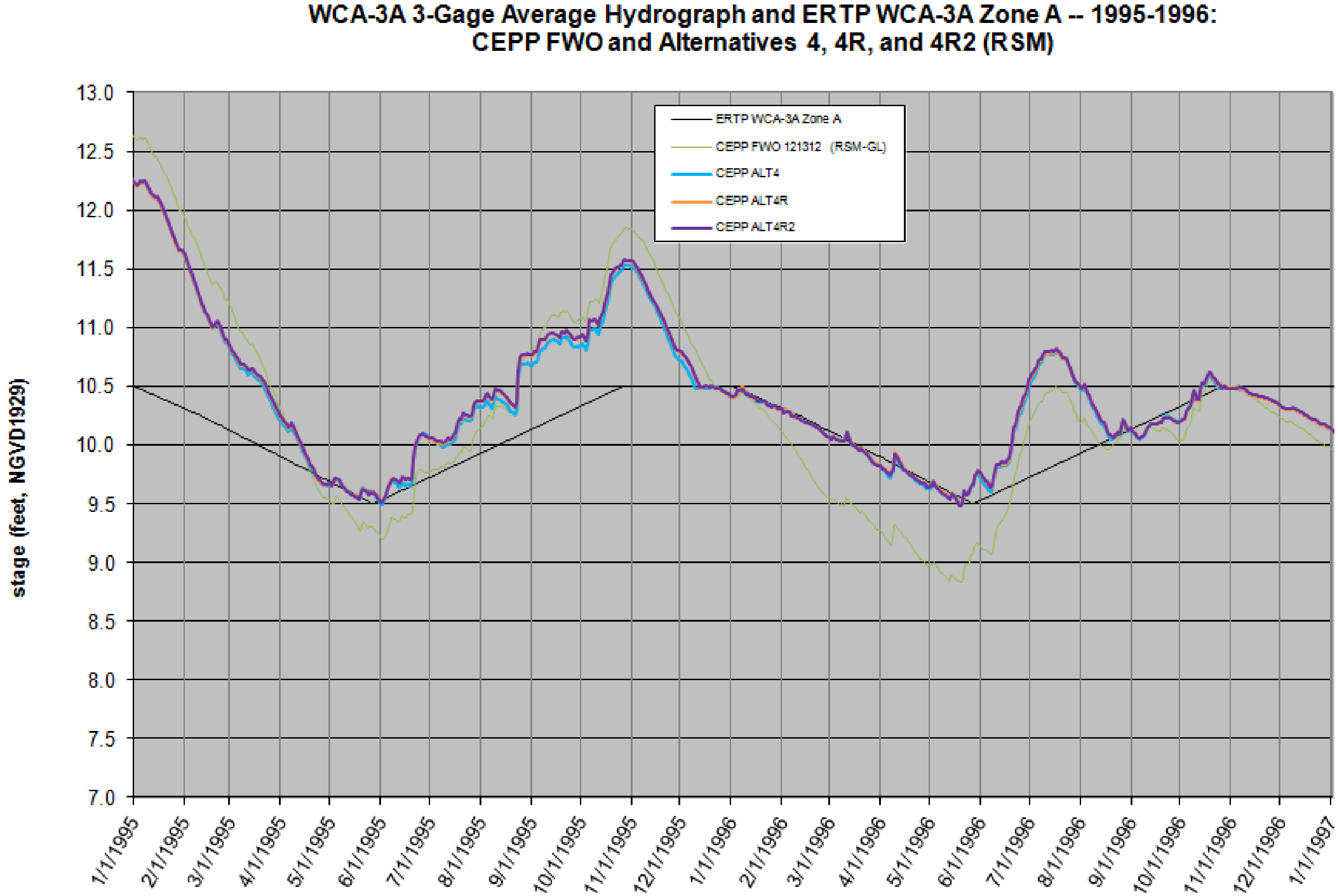


Figure 34: 1995-1996 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

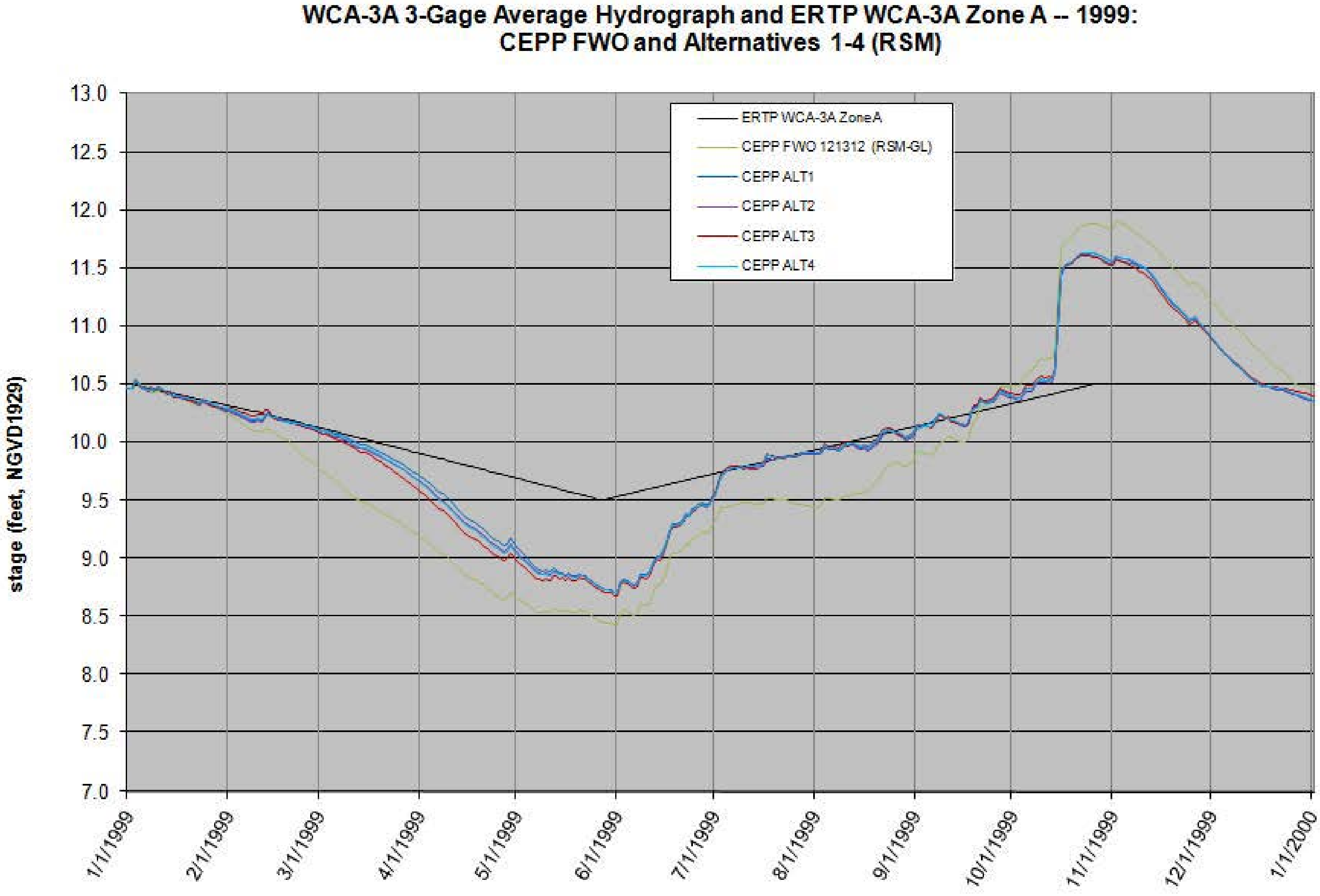


Figure 35: 1999 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4

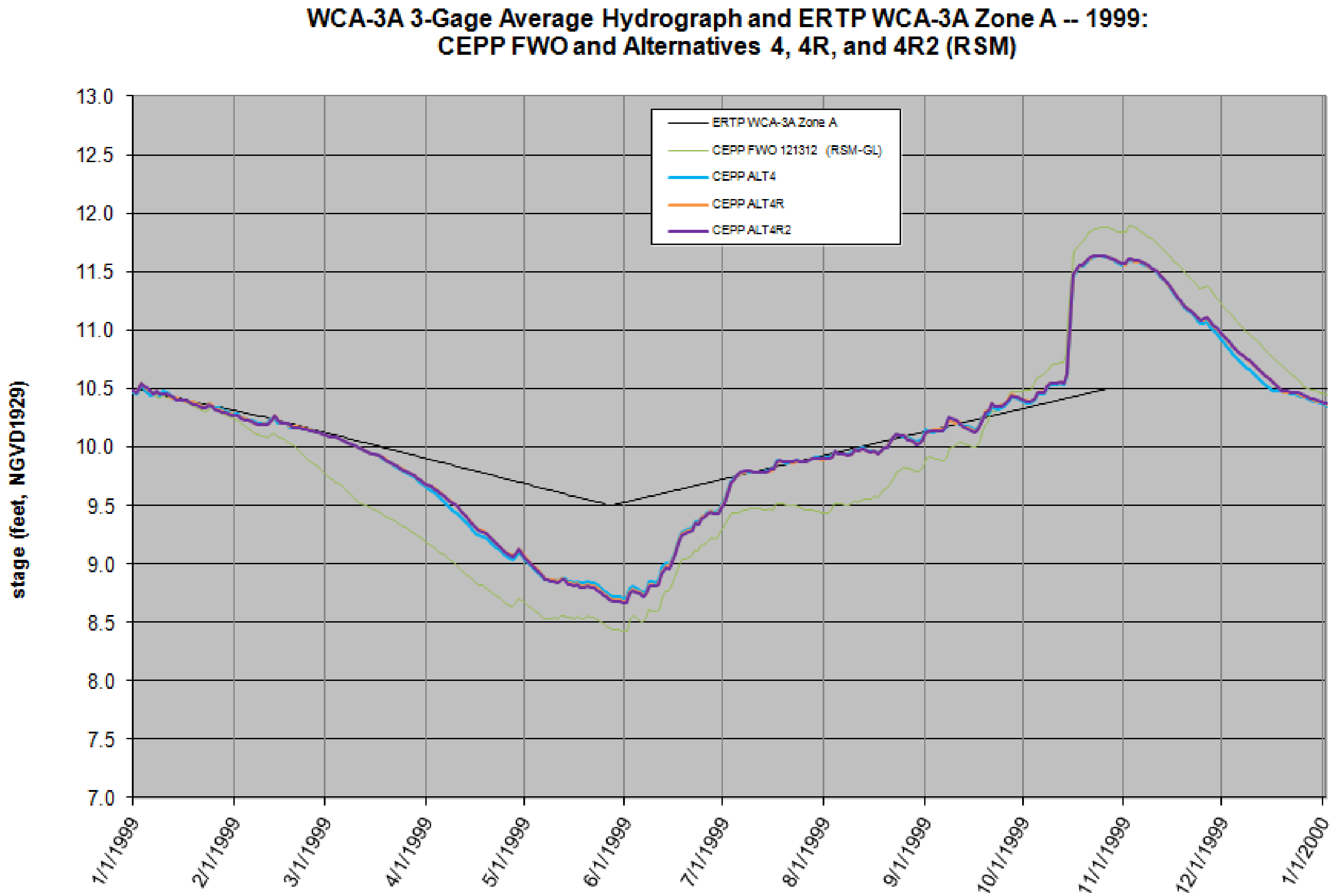


Figure 36: 1999 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 4,4R, and 4R2

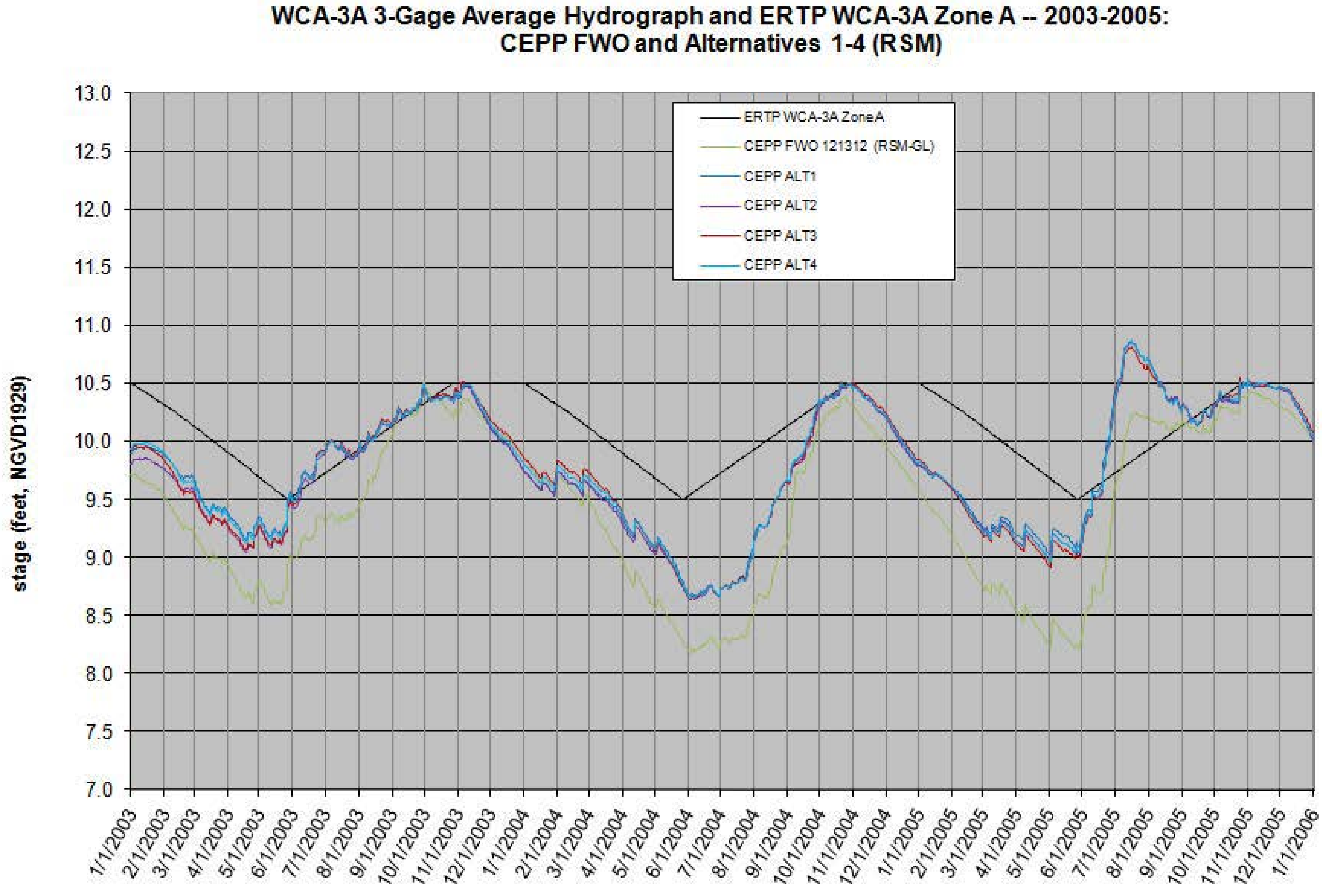


Figure 37: 2003-2005 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 1 through 4

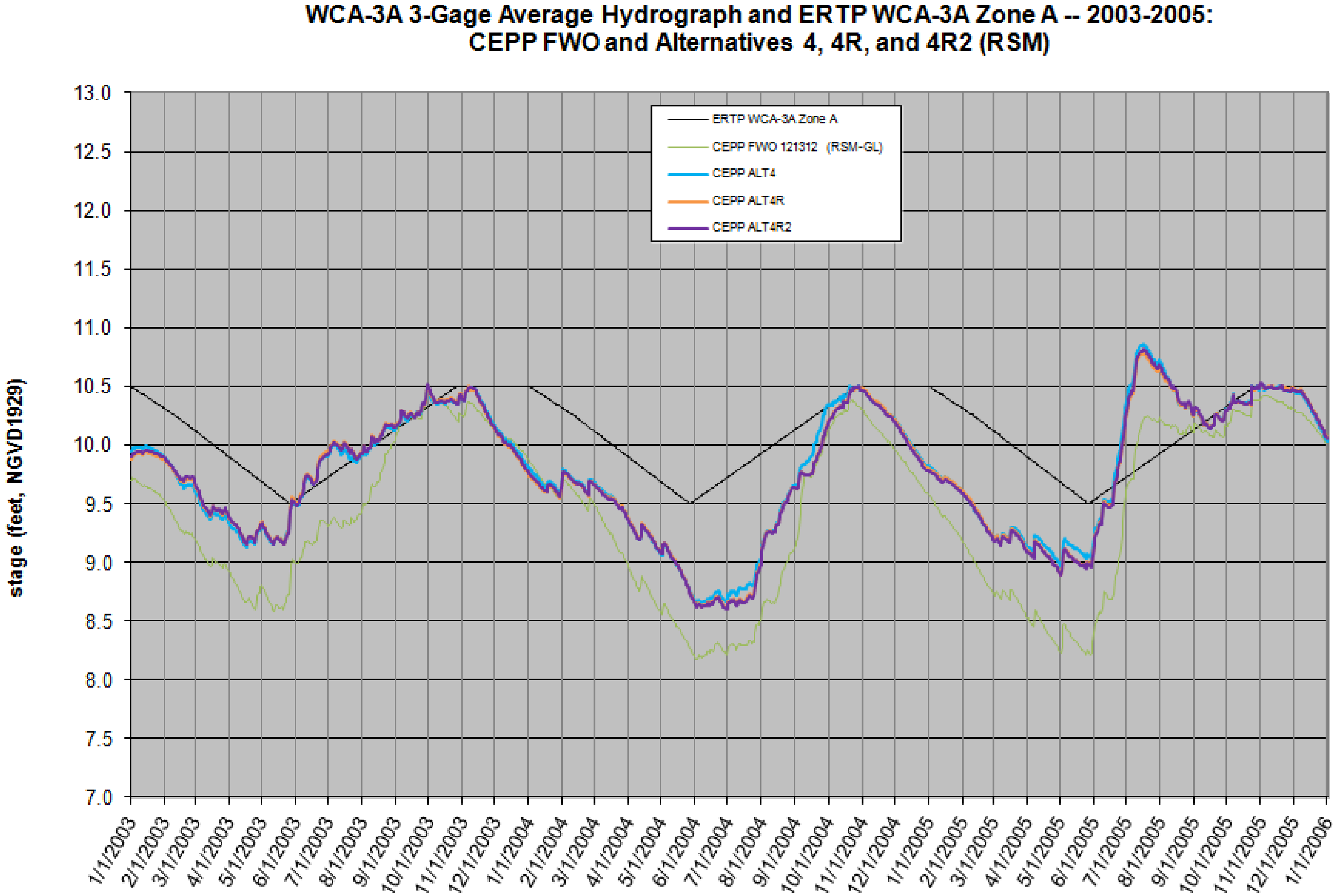


Figure 38: 2003-2005 WCA-3A 3-gage average hydrographs for CEPP FWO and CEPP Alternatives 4, 4R, and 4R2

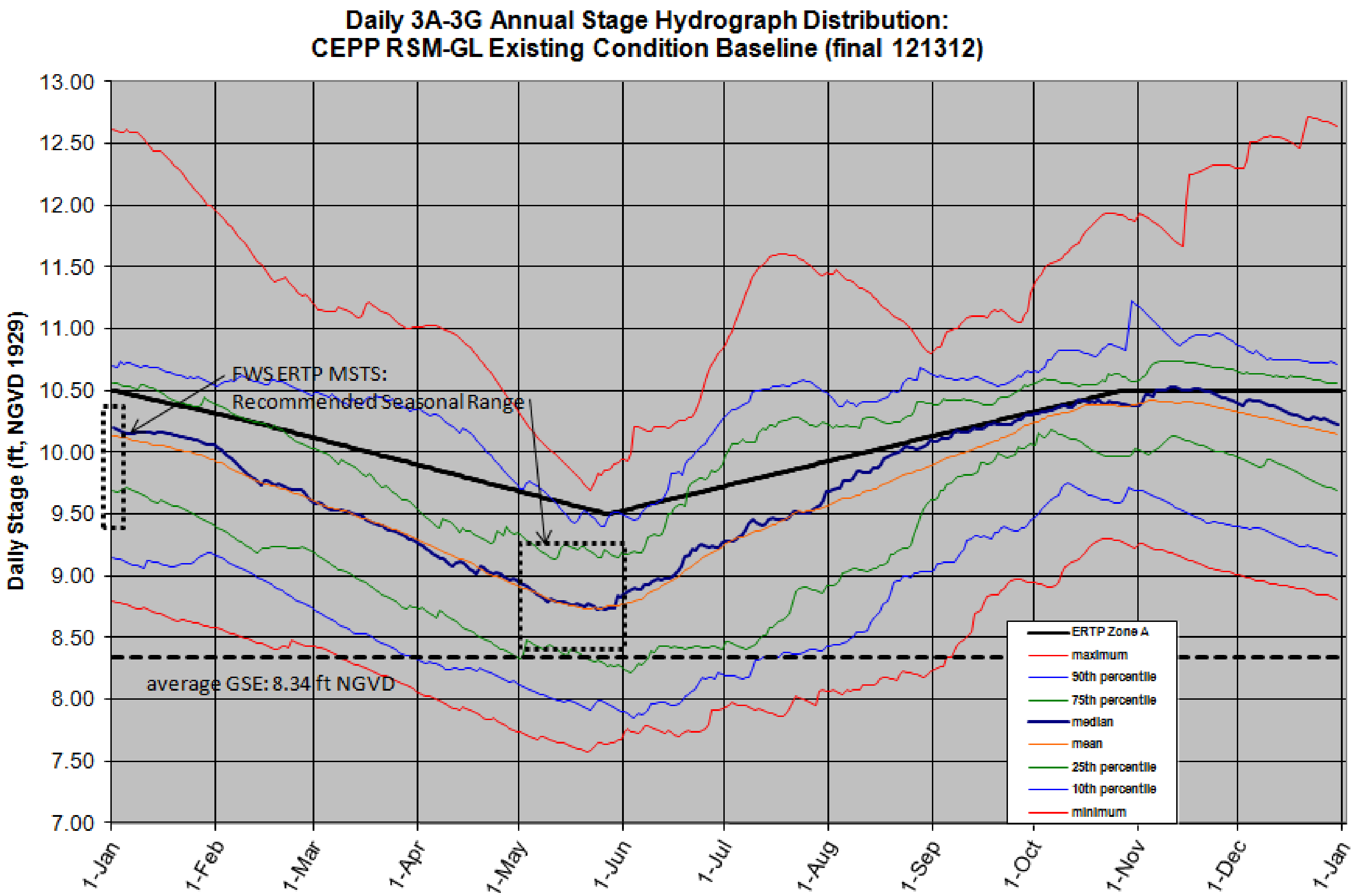


Figure 39: WCA-3A 3-gage average annual average stage hydrographs for CEPP ECB

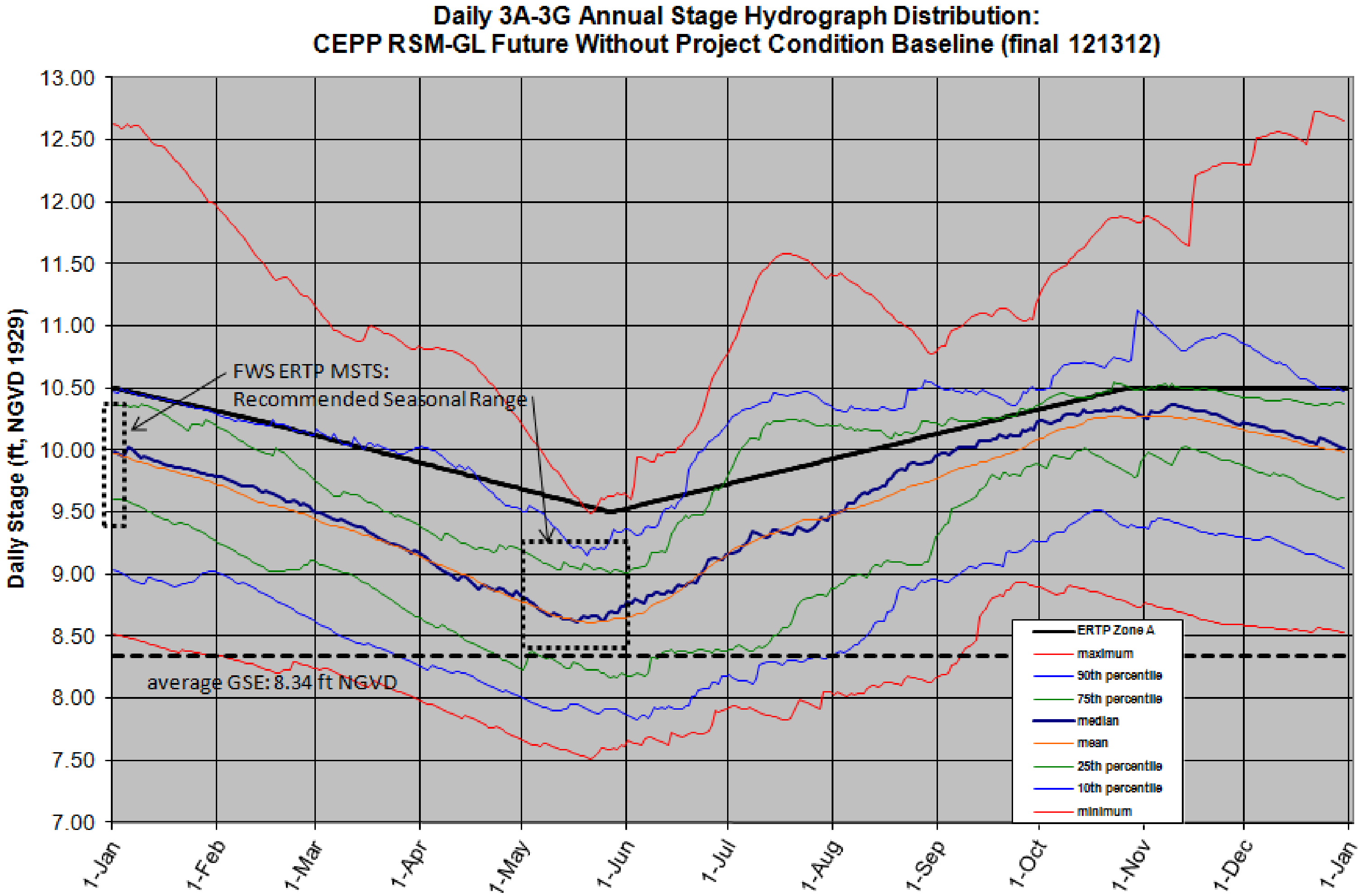


Figure 40: WCA-3A 3-gage average annual average stage hydrographs for CEPP FWO

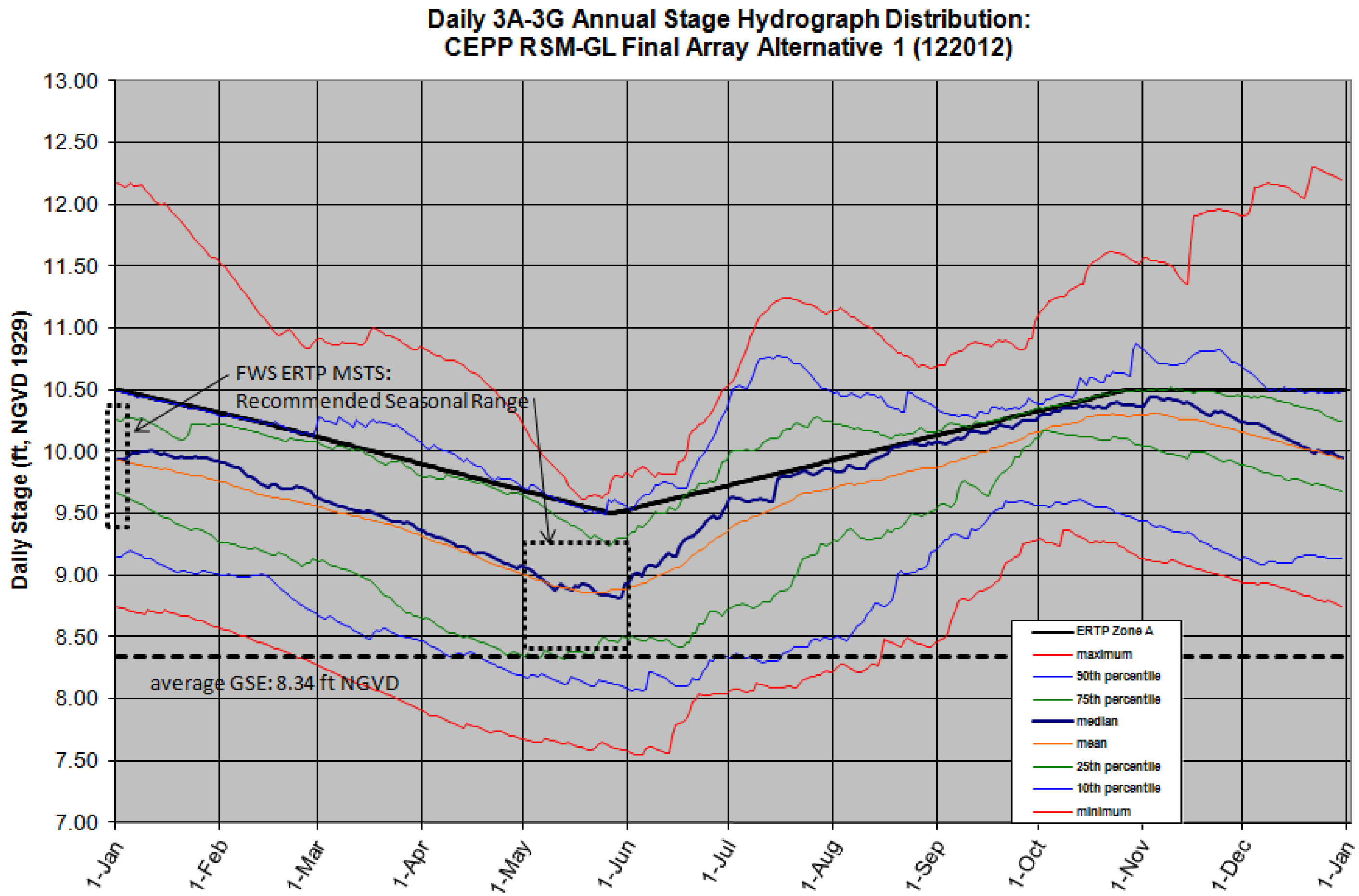


Figure 41: WCA-3A 3-gage average annual average stage hydrographs for CEPP Alternative 1

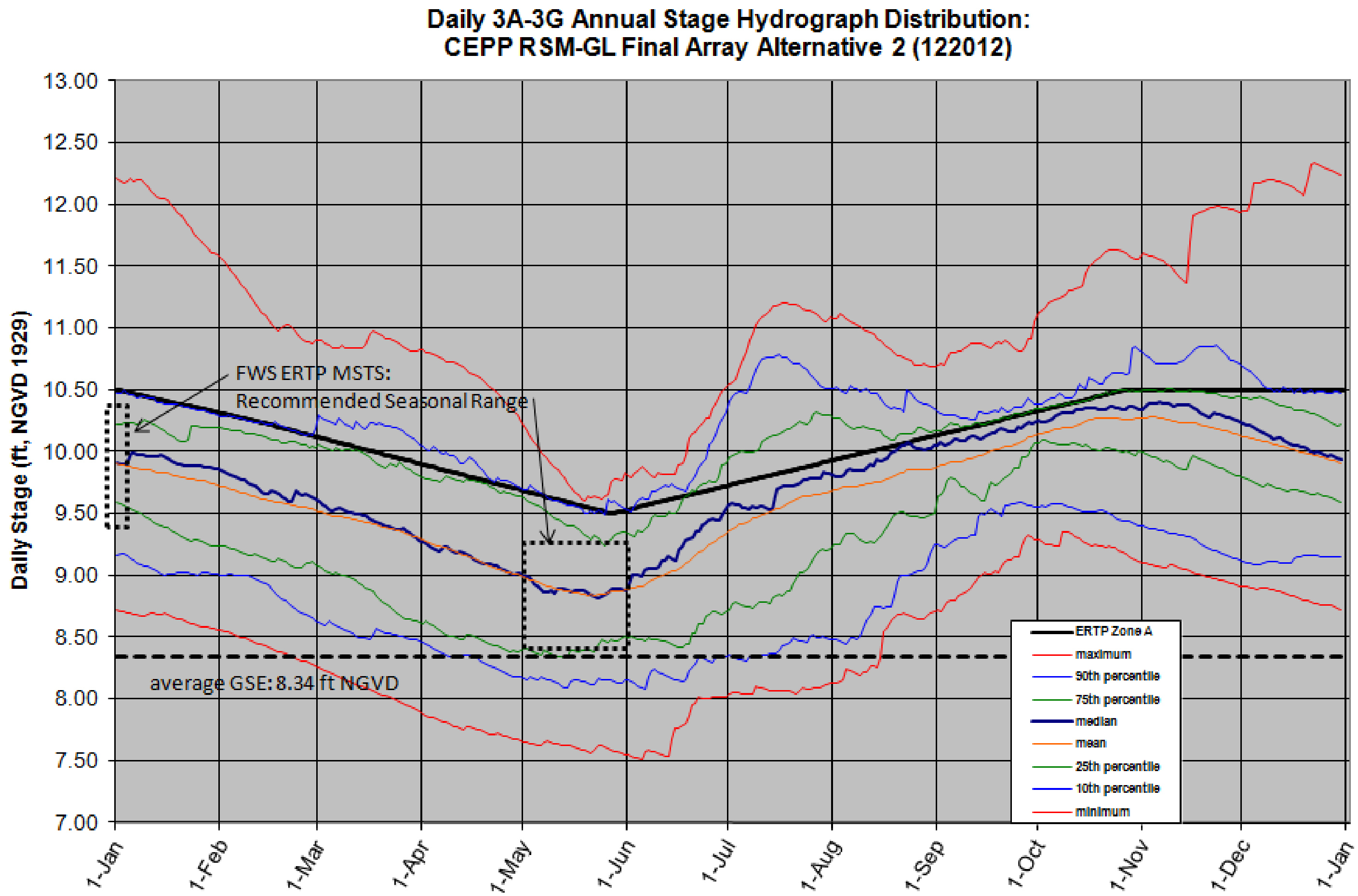


Figure 42: WCA-3A 3-gage average annual average stage hydrographs for CEPP Alternative 2

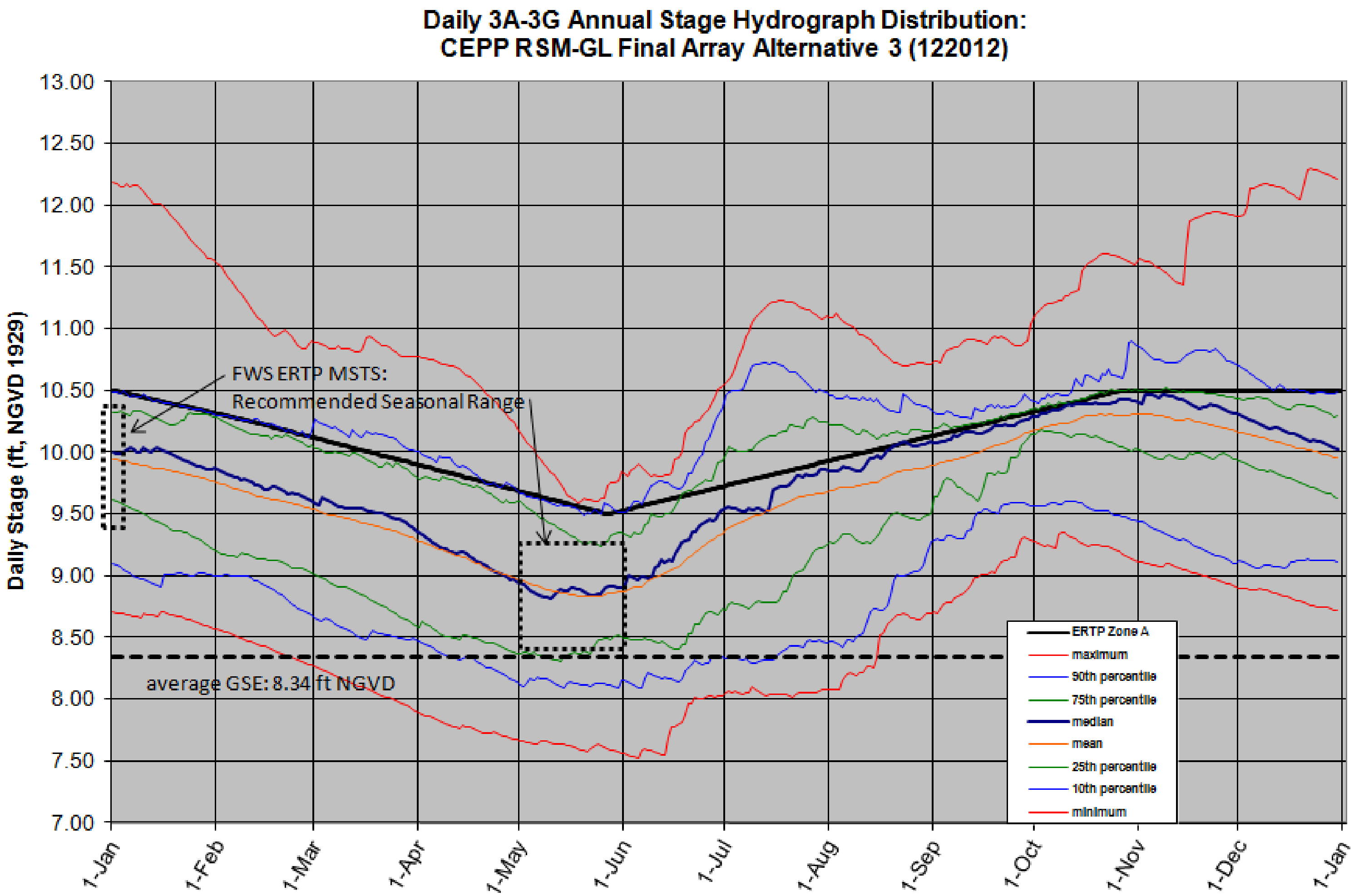


Figure 43: WCA-3A 3-gage average annual average stage hydrographs for CEPP Alternative 3

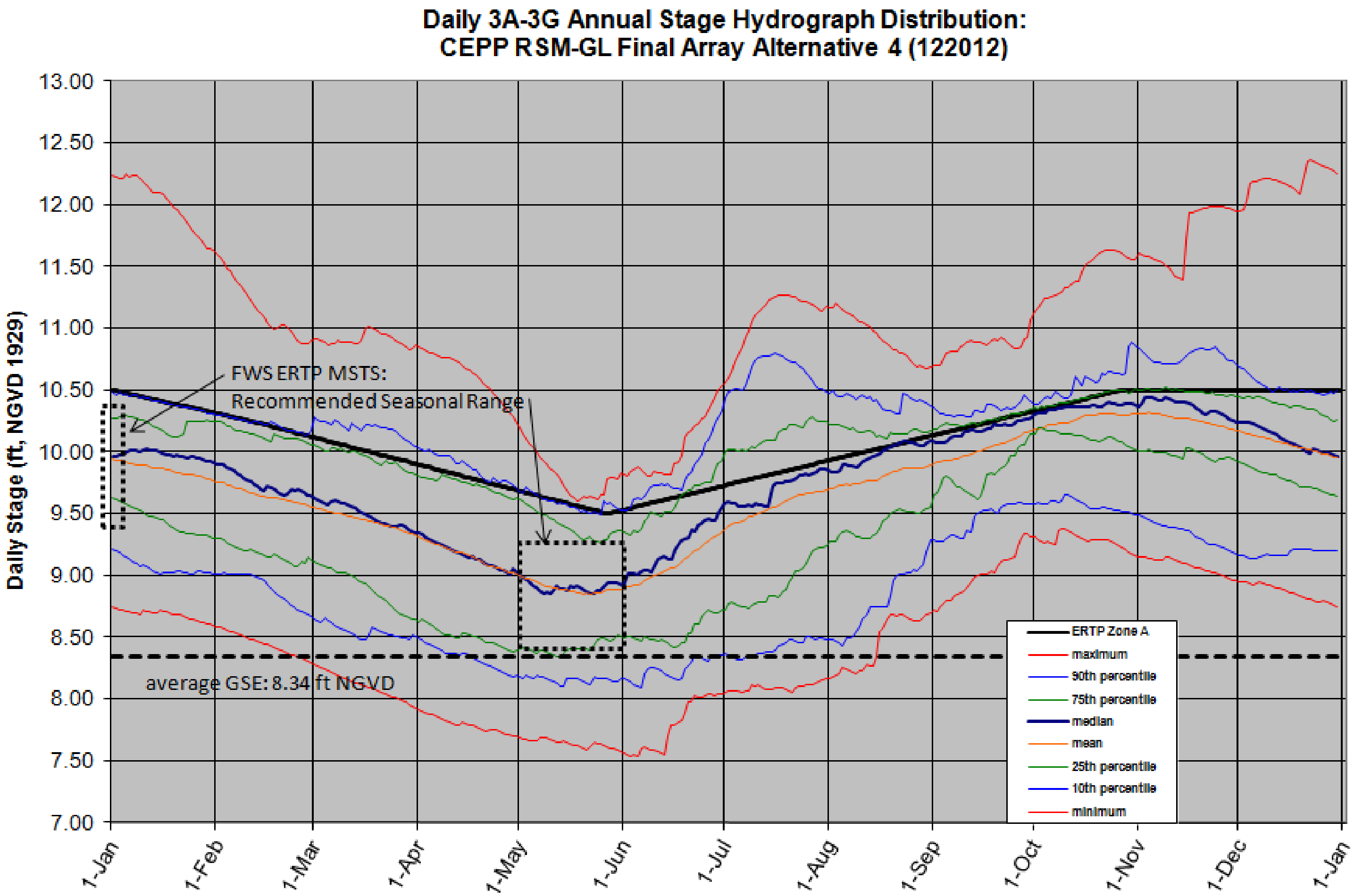


Figure 44: WCA-3A 3-gage average annual average stage hydrographs for CEPP Alternative 4

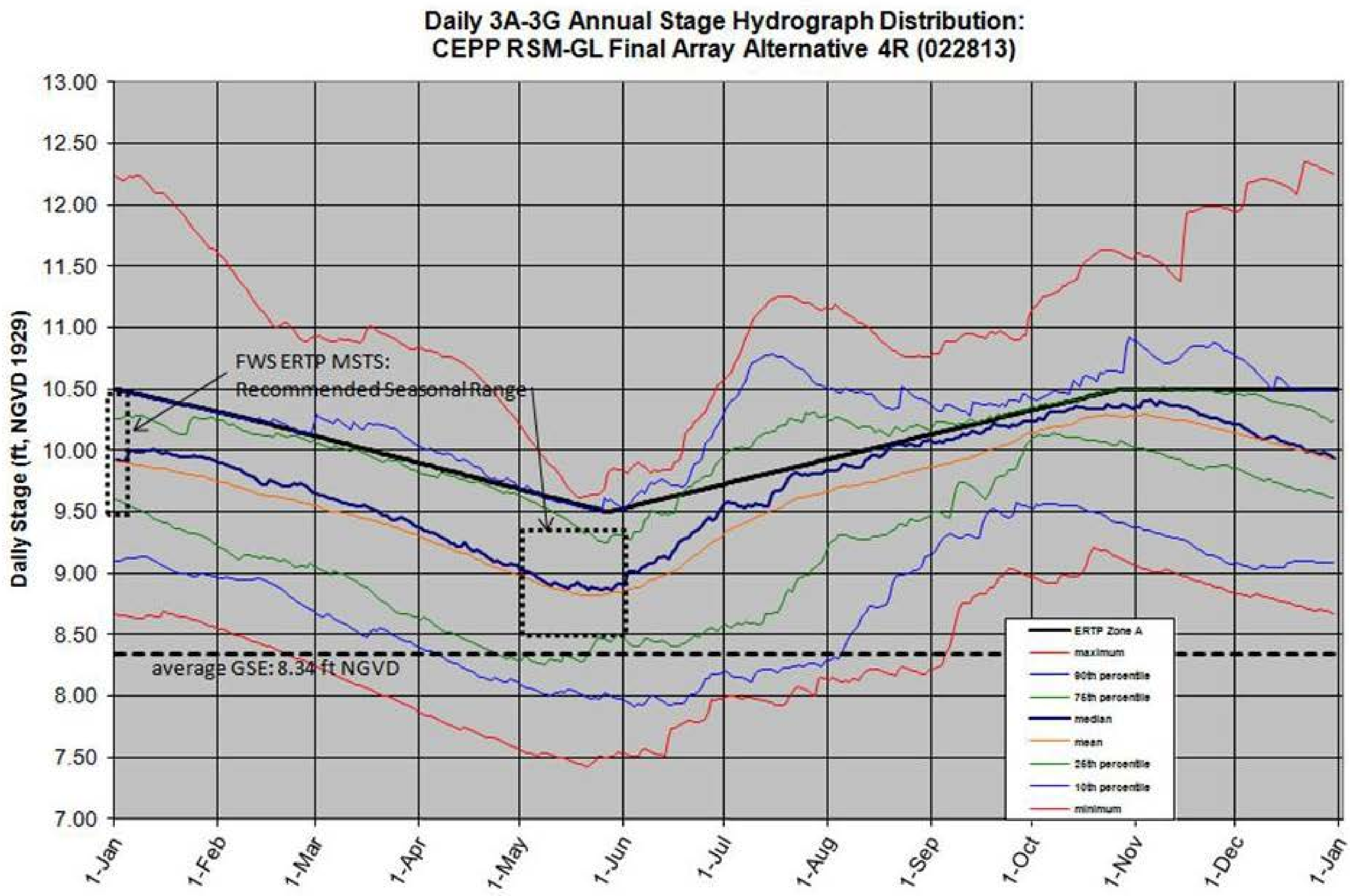


Figure 45: WCA-3A 3-gage average annual average stage hydrographs for CEPP Alternative 4R

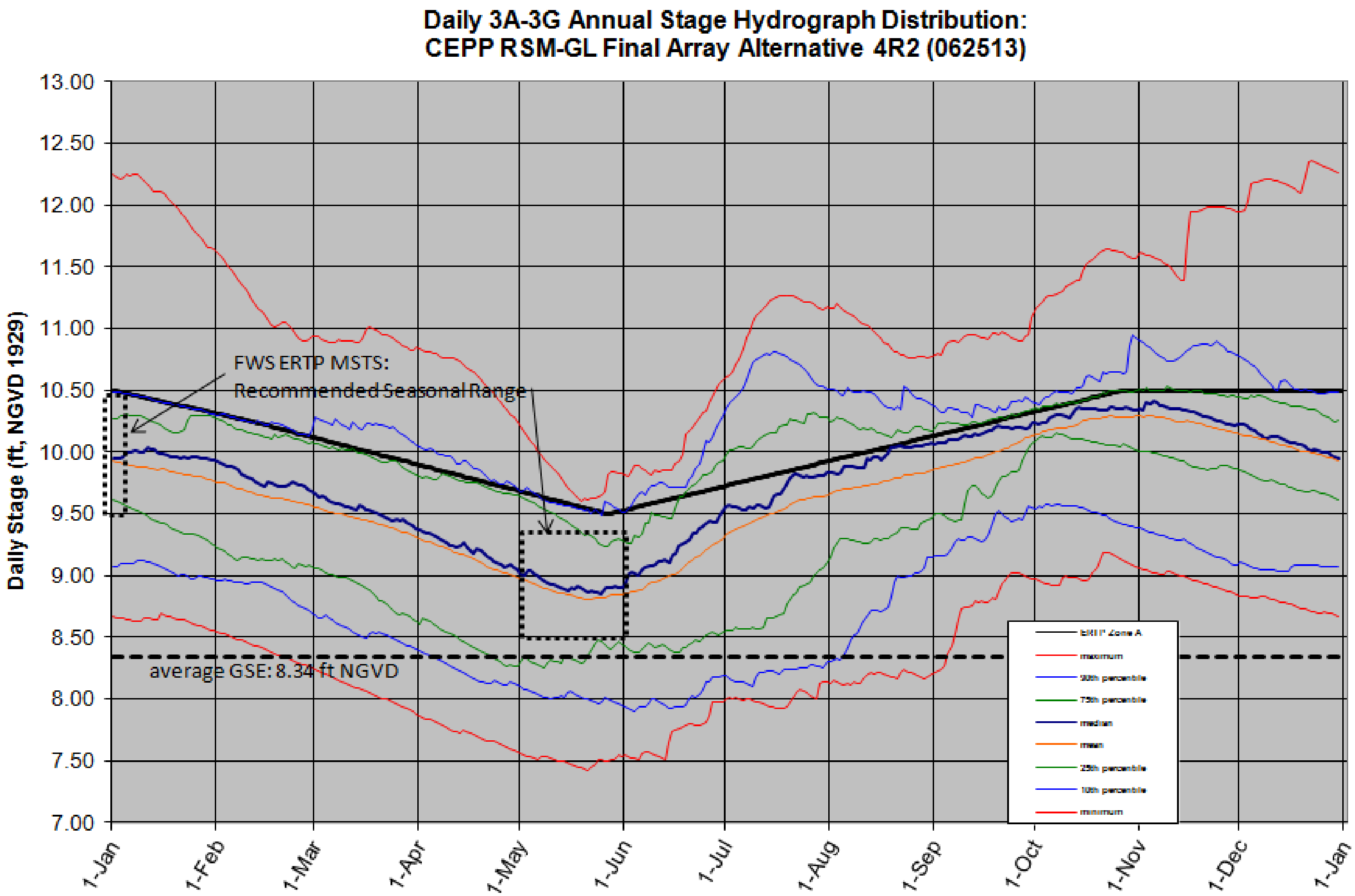


Figure 46: WCA-3A 3-gage average annual average stage hydrographs for CEPP Alternative 4R2

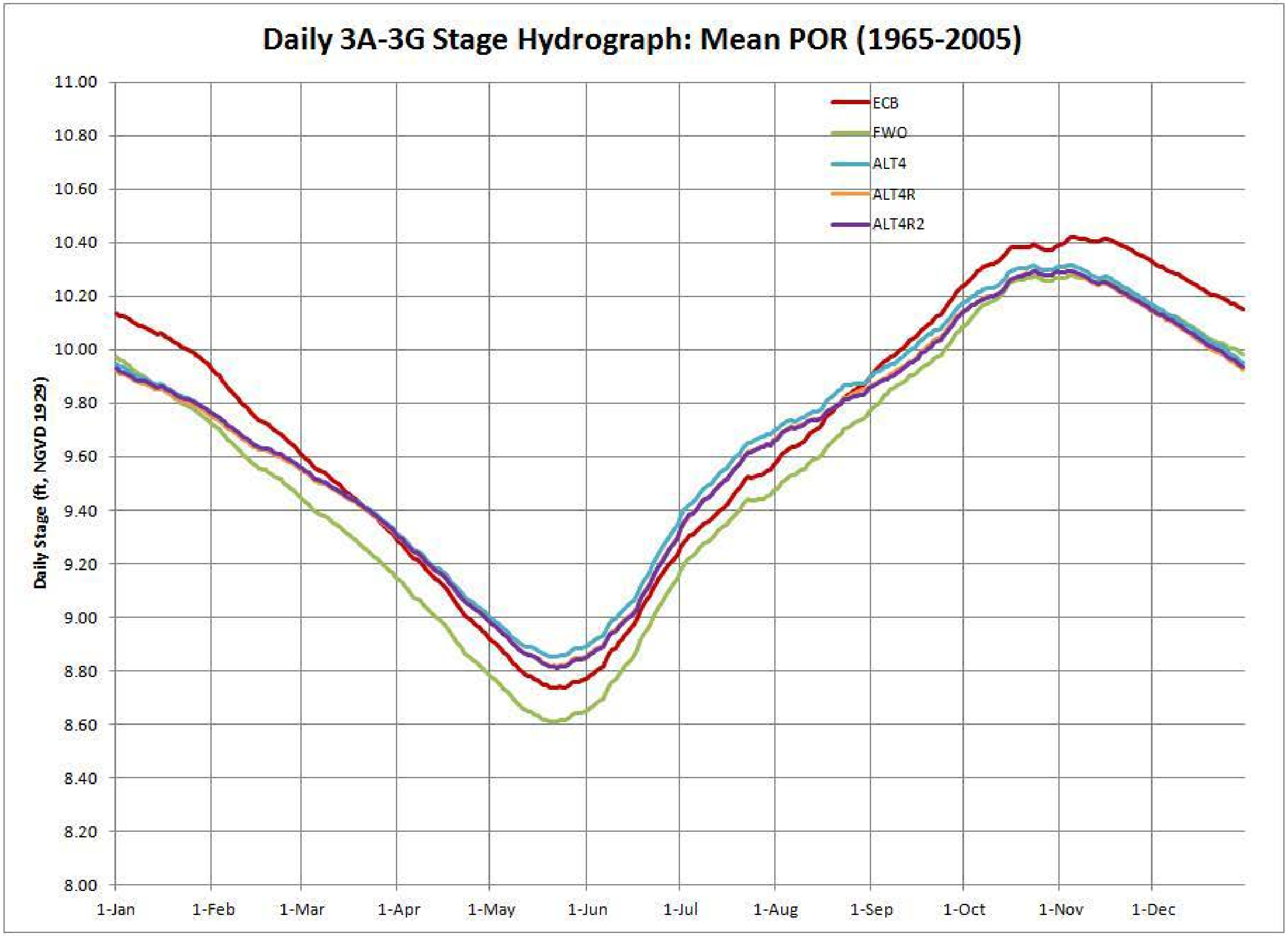


Figure 47: WCA-3A 3-gage average mean daily stage hydrographs for CEPP Alternatives 4, 4R, and 4R2

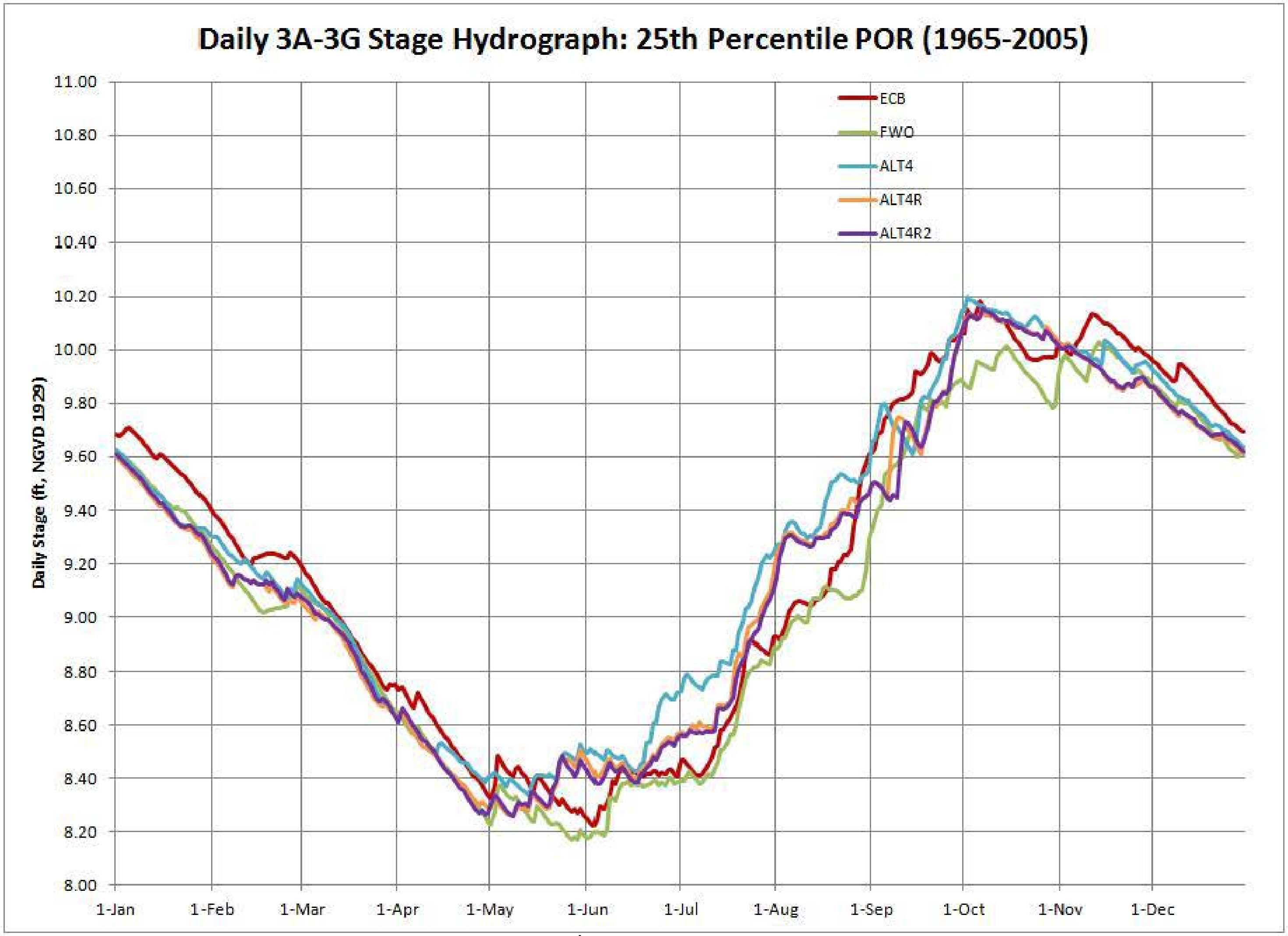


Figure 48: WCA-3A 3-gage average 25th percentile daily stage hydrographs for CEPP Alternatives 4, 4R, and 4R2

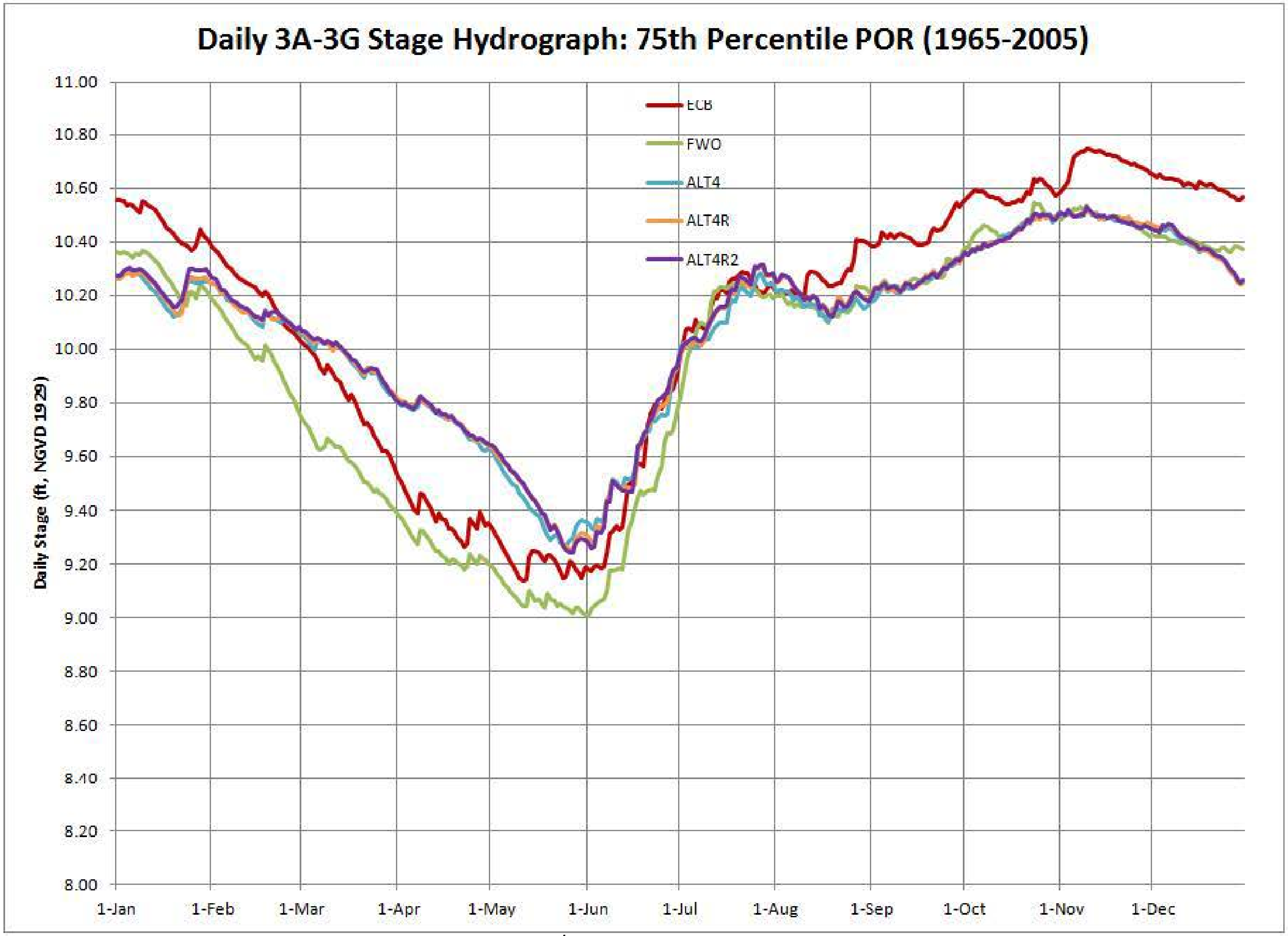


Figure 49: WCA-3A 3-gage average 75th percentile daily stage hydrographs for CEPP Alternatives 4, 4R, and 4R2

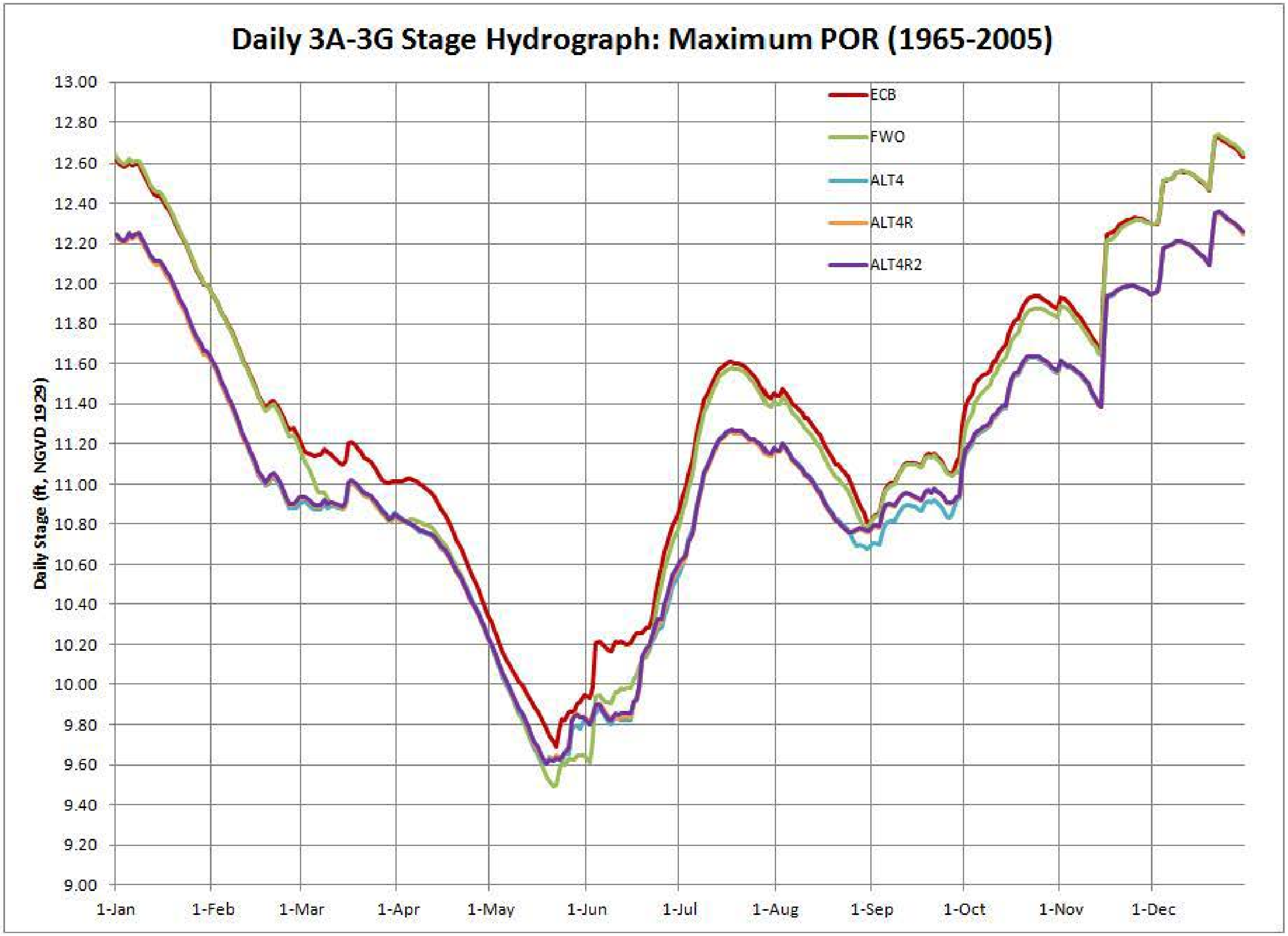


Figure 50: WCA-3A 3-gage average maximum daily stage hydrographs for CEPP Alternatives 4, 4R, and 4R2

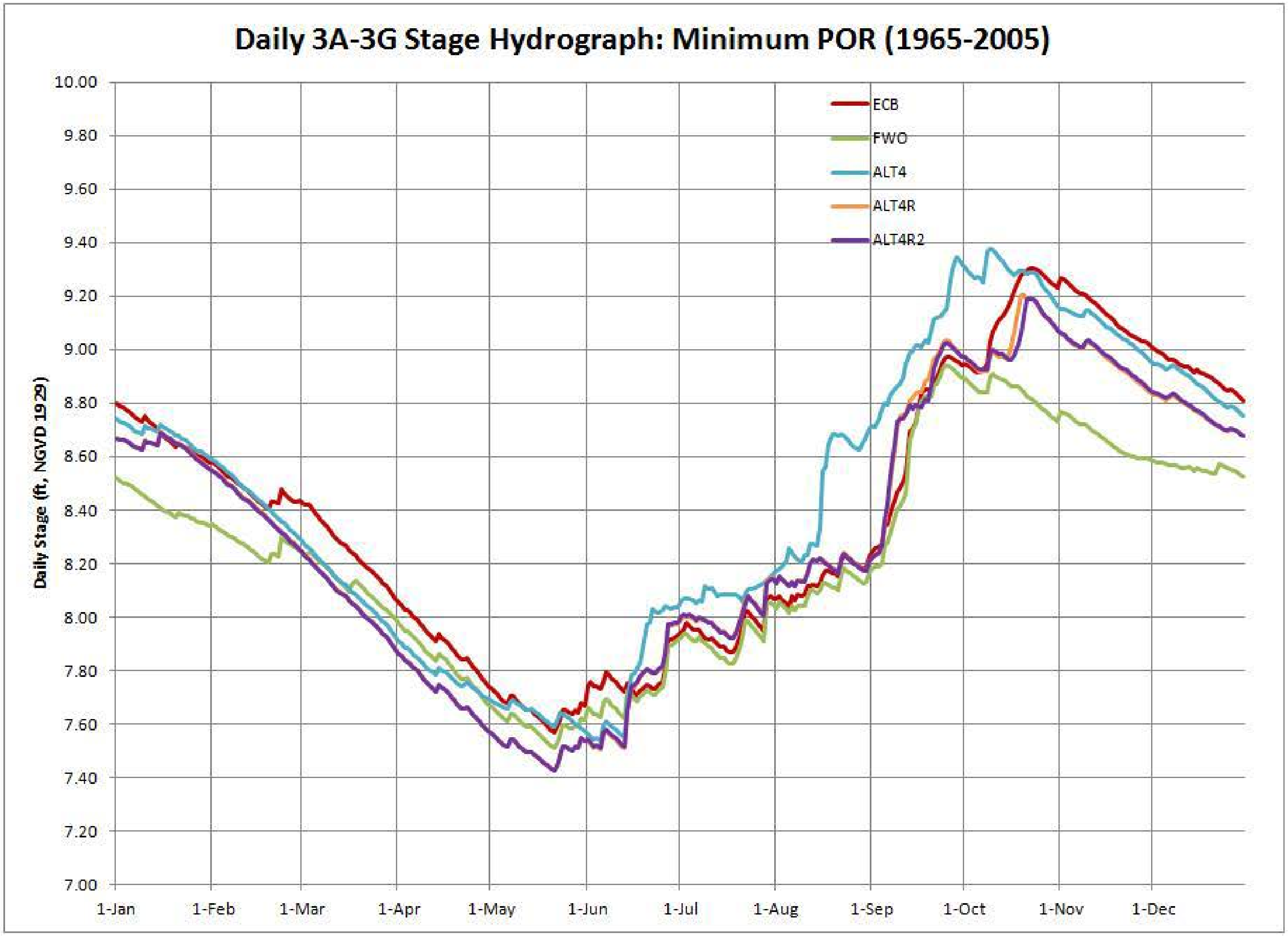


Figure 51: WCA-3A 3-gage average minimum daily stage hydrographs for CEPP Alternatives 4, 4R, and 4R2

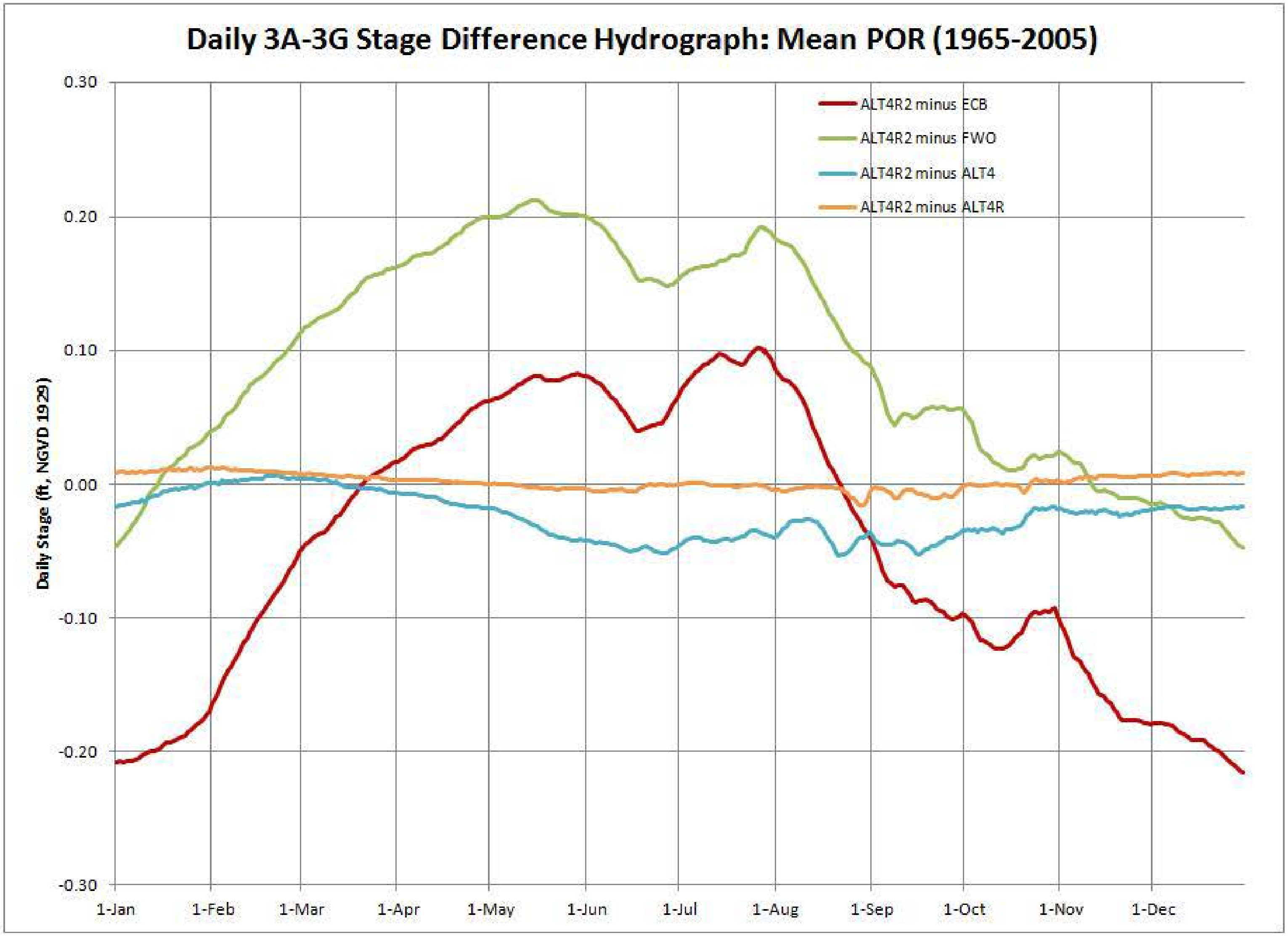


Figure 52: WCA-3A 3-gage average mean daily stage difference hydrographs, compared to Alternative 4R2

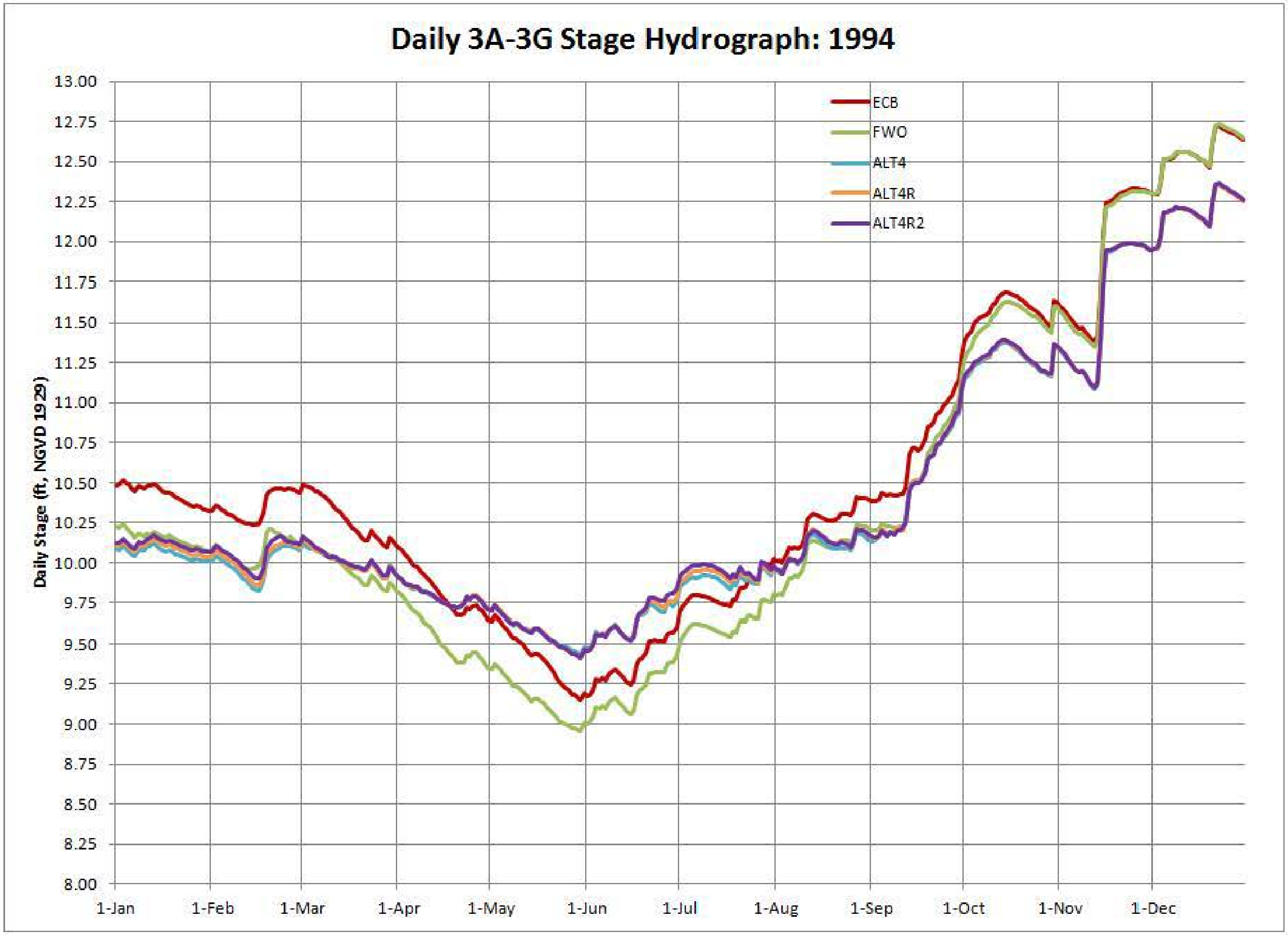


Figure 53: 1994 WCA-3A 3-gage average daily stage hydrograph for CEPP Alternatives 4, 4R, and 4R2

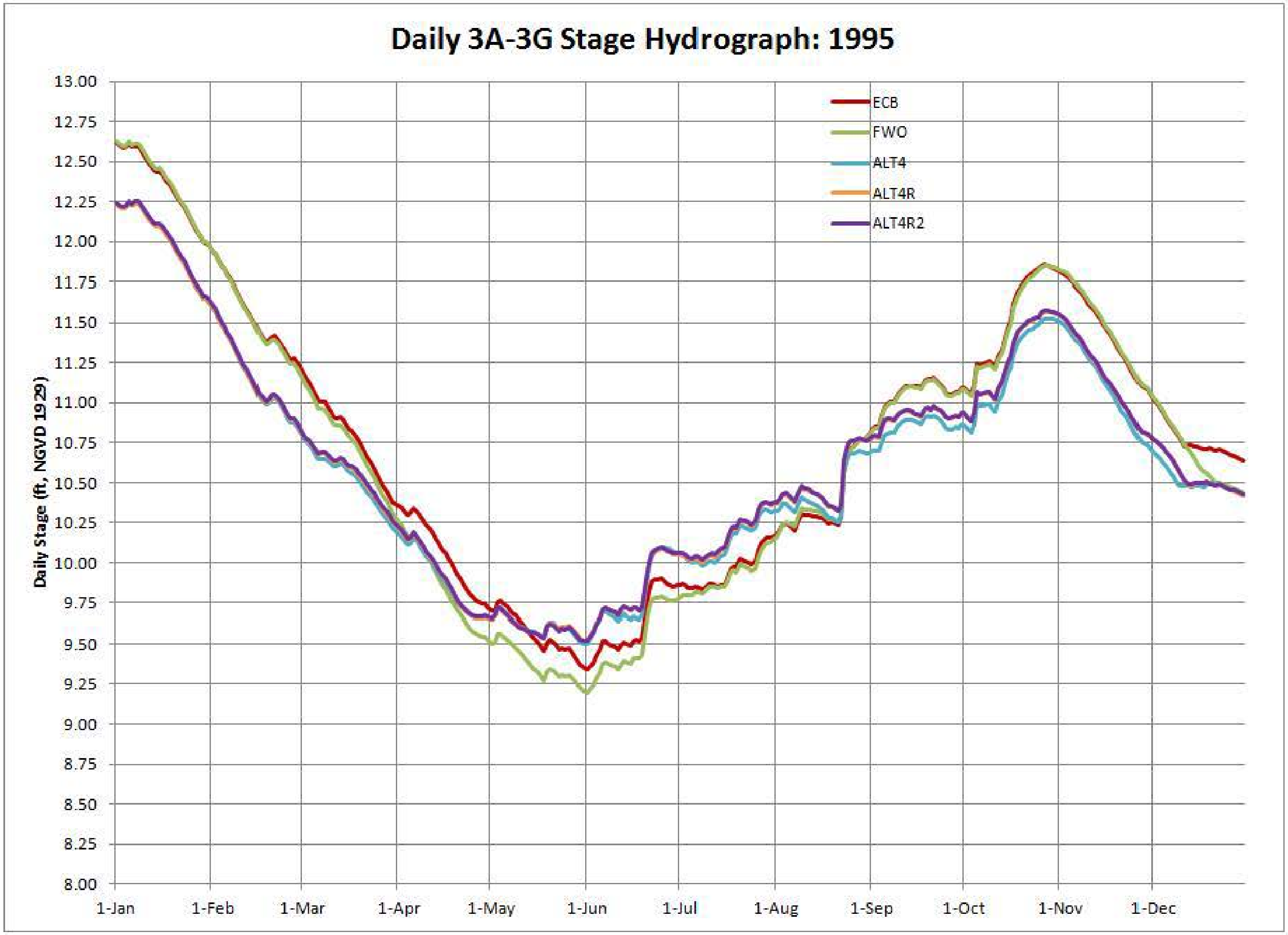


Figure 54: 1995 WCA-3A 3-gage average daily stage hydrograph for CEPP Alternatives 4, 4R, and 4R2

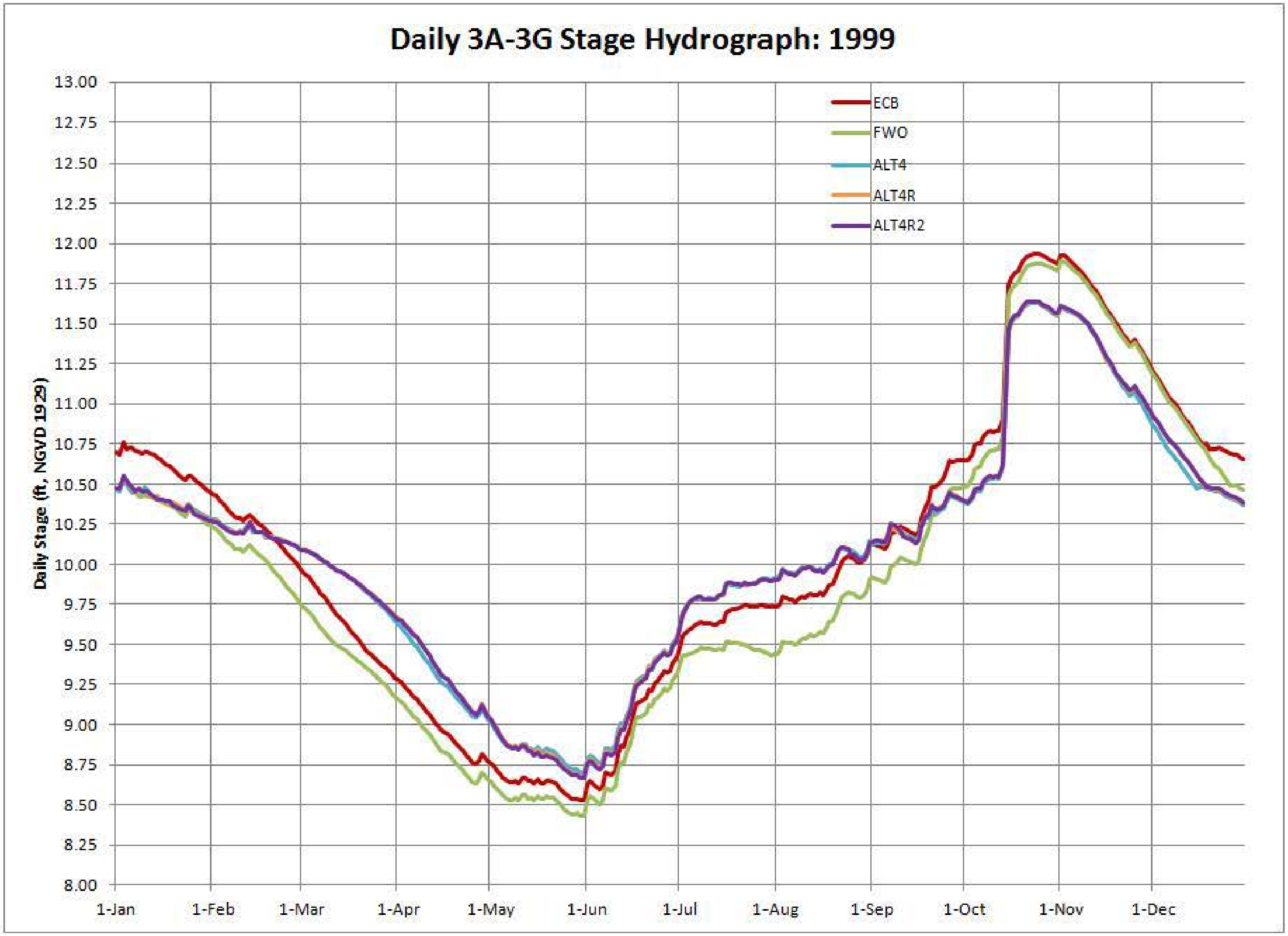


Figure 55: 1999 WCA-3A 3-gage average daily stage hydrograph for CEPP Alternatives 4, 4R, and 4R2

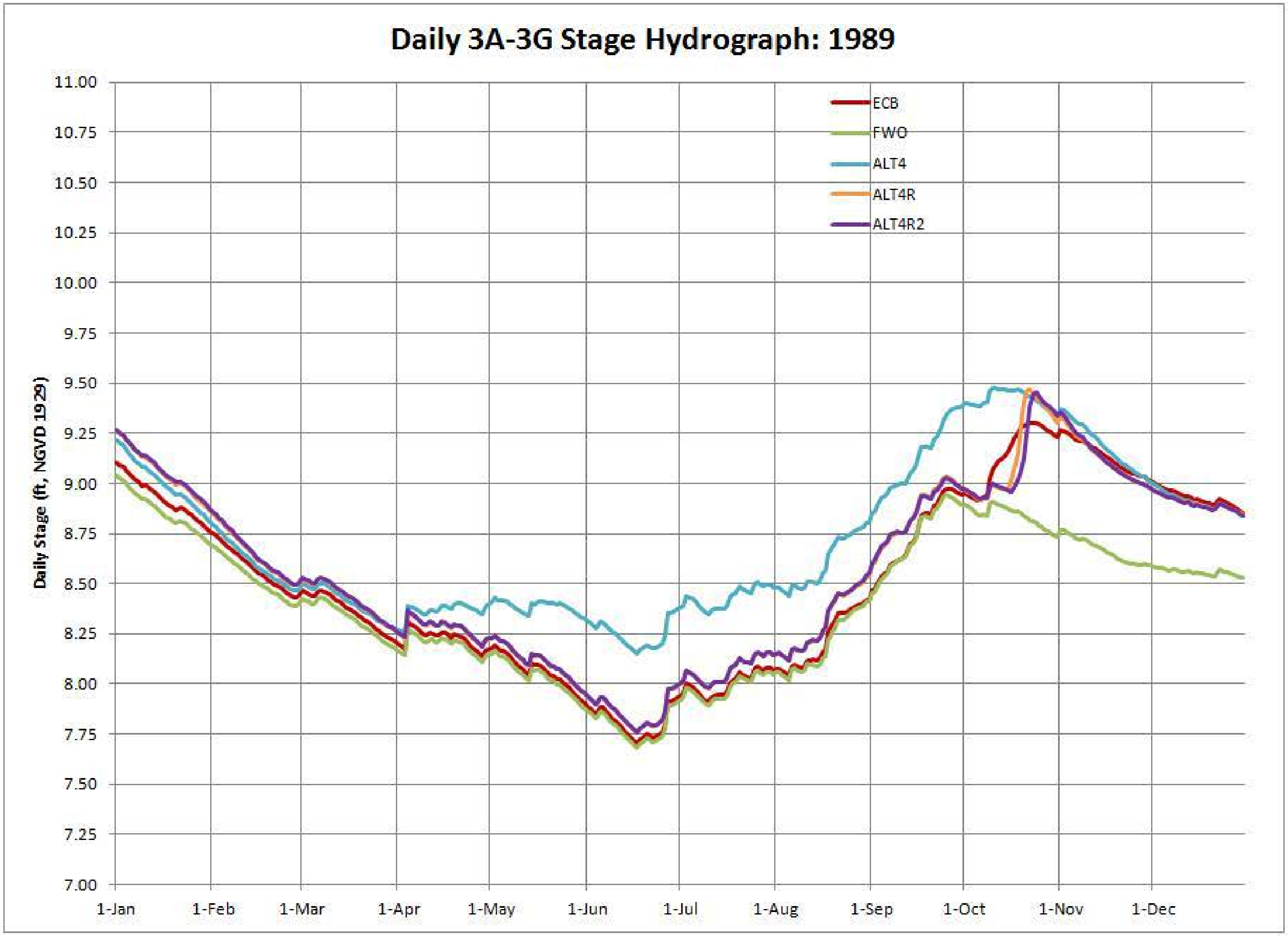


Figure 56: 1989 WCA-3A 3-gage average daily stage hydrograph for CEPP Alternatives 4, 4R, and 4R2

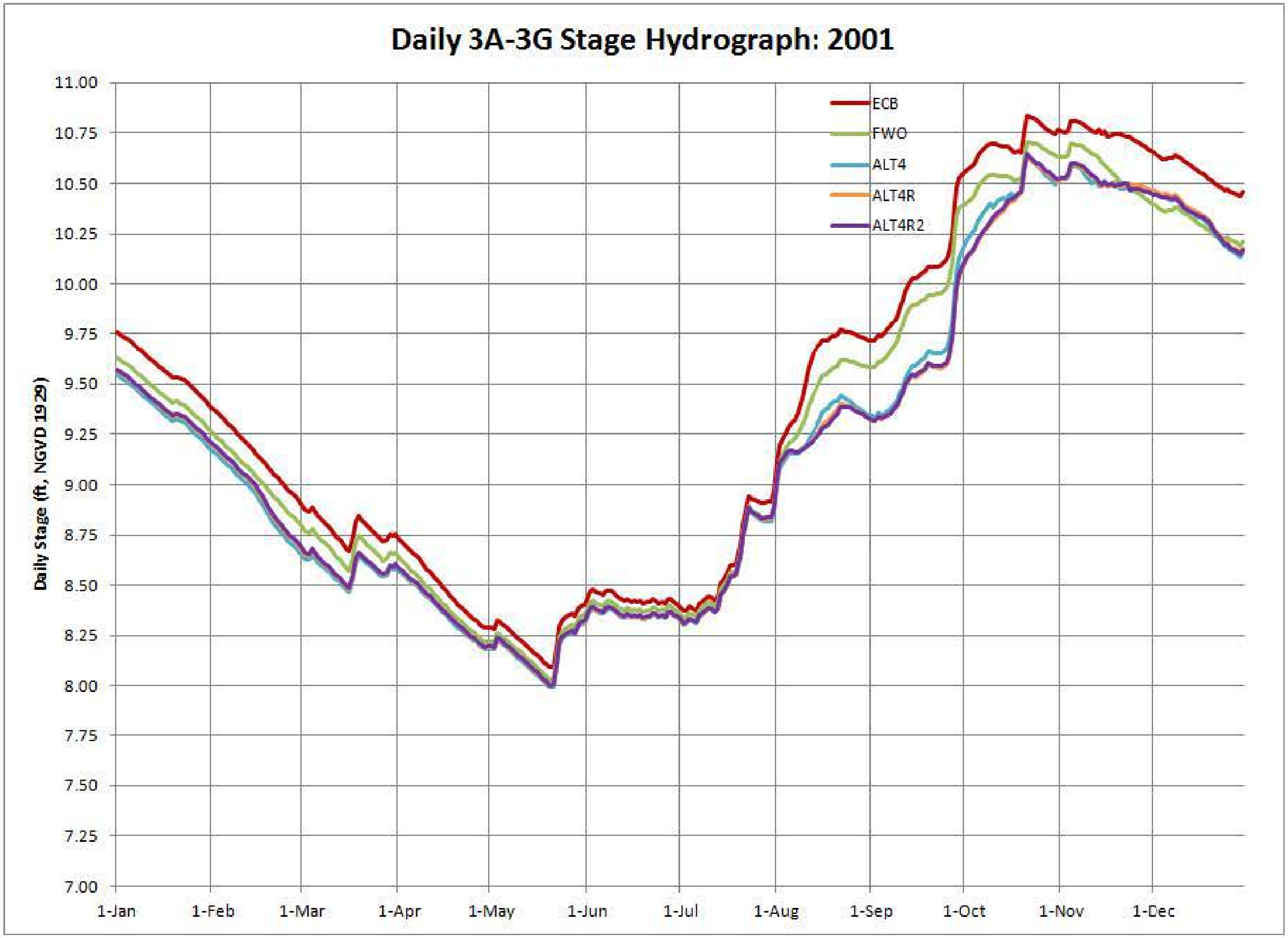


Figure 57: 2001 WCA-3A 3-gage average daily stage hydrograph for CEPP Alternatives 4, 4R, and 4R

3.2.2. WCA-3B Design Considerations

Subsequent to completion of the L-67A Levee in 1962 (the adjacent L-67C Levee was completed in 1966), WCA-3B water levels have been highly managed. The S-151 gated culvert (1105 cfs design capacity) currently provides the only structural connection between WCA-3A and WCA-3B. The SPF stage for WCA-3B, based on Site 71 (refer to the Figure 7 map), was initially established in the 1960 GDM for WCA-3 (C&SF Part 1, Supplement 33) at 8.50 feet NGVD based on an assumed 5-day, 16.5-inch rainfall event; detailed SPF flood routing information for WCA-3B is not provided in the GDM. Starting in 1985, the C&SF Experimental Program for Water Deliveries to ENP established S-151 operational criteria that discontinued S-151 regulatory releases from WCA-3A if stages at Site 71 exceed 8.5 feet NGVD. The Site 71 constraint at 8.5 feet NGVD was also used for the 1994-1995 L-67 gap tests, which were conducted as design tests for the MWD to ENP Project. The IOP and ERTP WCA-3A Regulation Schedules specify operation of S-151 for water supply only during Column 1 operations (no WCA-3A regulatory releases to the South Dade Conveyance System (SDCS)) and S-151 regulatory inflows to WCA-3B during Column 2 operations (WCA-3A regulatory releases to the SDCS), contingent on the Site 71 stage being below 8.5 feet NGVD.

The USACE has not conducted a comprehensive review of the previously-established SPF stages for WCA-3B, pending consideration of modified inflow infrastructure for WCA-3B. SFWMM modeling conducted for the 1993 MWD to ENP Feature Design Memorandum (FDM), based on the 1992 MWD GDM default operational plan, identified a revised SPF stage of 11.6 feet NGVD at Site 71 for the MWD Project condition; however, despite subsequent multiple interagency efforts, a final configuration for WCA-3B inflow structures and an associated MWD operational plan, has not been identified prior to the conclusion of CEPP formulation efforts.

Concurrent with CEPP alternative formulation and modeling efforts, EN-W conducted a review of WCA-3B high water levels compared to the WCA-3B design criteria and independent of any previous SPF stage considerations. WCA-3B is currently bounded by the L-29 Levee (Section 3) to the south, the L-67A Levee and the L-67C Levee to the west, and the L-30 Levee to the east; the design grades for these WCA-3B perimeter levees range between 13.0 feet NGVD for the L-29 Levee (note: typical sections range from 13.5-17.5 feet NGVD, due to subsequent stockpiling of spoil material from L-29 Canal improvements, and all L-29 Section 3 Levee sections meet or exceed the design grade) to 20.0 feet NGVD for the L-30 Levee (the design grades for the L-67A and L-67C Levees are 17.5 and 12.5 feet NGVD, respectively), such that the L-29 Levee design grade represents the limiting factor for peak WCA-3B stages for CEPP. Stage duration curves for the CEPP ECB, the CEPP FWO, and the CEPP final array alternatives (including the operational modifications of the TSP) are provided in Figures 58 through 65 for the two RSM-GL monitoring gage locations within WCA-3B at Site 71 and Shark-1 (also alternatively referred to as SRS-1) that are produced with the model standard output information; corresponding RSM-GL model GSE elevations for these gages are 6.64 and 6.61 feet NGVD, respectively. Annual stage hydrograph statistical distribution plots for Site 71, which is currently utilized for WCA-3B operational management, are provided in Figures 66-68 to facilitate comparisons of intra-annual stage variability between the CEPP ECB baseline condition, the CEPP FWO baseline condition, and the TSP Alternative 4R2. For the 41-year period of simulation, the graphics illustrate the maximum and minimum stage, 90th and 10th percentile stages, 75th and 25th percentile stages, median stage, and mean stage at a daily time step. For the CEPP alternatives, peak stages within WCA-3B (outside of the Blue Shanty Flow-way in Alternatives 4, 4R, and 4R2) ranged between 9.22-

9.62 feet NGVD at Site 71 and between 9.22-9.65 feet NGVD at Shark-1. WCA-3B peak stages for the CEPP alternatives are approximately 0.15-0.60 feet greater than the CEPP ECB and CEPP FWO baselines; however, the WCA-3B peak stages for the CEPP alternatives remain approximately 3.4-3.8 feet below the L-29 Section 3 design grade of 13.0 feet NGVD. The SPF rainfall for WCA-3B is approximately 1.5 feet (17.5 inches; based on the C&SF definition of 125% of the localized 3-day, 100-year maximum rainfall event of 14 inches (SFWMD Technical Publication EMA #390, January 2001)). Based on EN-W assessment of these maximum simulated WCA-3B peak water depths of slightly more than 3.0 feet for the final array of alternatives, maximum wind and wave run-up potentials would not be expected to exceed 1-2 feet. For this preliminary EN-W assessment (further analysis will be conducted during PED), a presumed worst-case scenario was defined with peak CEPP stages exacerbated by the additional SPF rainfall and maximum wind and wave run-up depths. Under the assumptions for this worst-case scenario, the L-29 Section 3 Levee would not be expected to be overtopped with the simulated stages for Alternatives 1, 4, 4R, or 4R2; potential overtopping under this worst-case scenario would only occur for the relatively higher WCA-3B stages simulated with Alternative 2 and Alternative 3, at the two lowest elevation points along the L-29 Section 3 Levee. Although Alternative 2 and Alternative 3 were not identified as the TSP, it is noted that potential WCA-3B depths in the range of those contemplated with Alternative 2 and Alternative 3 would most likely require additional analyses during PED and/or minor improvements to the L-29 Levee Section 3.

For CEPP TSP alternative 4R2, however, peak stages within WCA-3B (outside of the Blue Shanty Flow-way in Alternative 4R2) were 9.25 and 9.24 feet NGVD at Site 71 and Shark-1, respectively, or approximately 0.20 feet greater than the CEPP ECB and CEPP FWO baselines (9.05-9.06 feet NGVD); however, the WCA-3B peak stages for the CEPP TSP plan remains approximately 3.75 feet below the L-29 Section 3 design grade of 13.0 feet NGVD. The SPF rainfall for WCA-3B is approximately 1.5 feet. Based on EN-W assessment of these WCA-3B peak water depths less than 3 feet (2.61-2.63 feet peak depth for Alternative 4R2 stages), maximum wind and wave run-up potentials would not be expected to exceed 1-2 feet. For this preliminary EN-W assessment (further analysis will be conducted during PED), a presumed worst-case scenario was defined for the CEPP TSP plan, with peak Alternative 4R2 stages exacerbated by the additional SPF rainfall and maximum wind and wave run-up depths. Under this assumed worst-case scenario (9.25 feet NGVD stage + 1.5 feet SPF rainfall + 2.0 feet run-up potential), the L-29 Section 3 Levee would not be overtopped at the two lowest elevation points (with approximately 0.25 feet of remaining freeboard, compared to the minimum L29 Section 3 Levee elevation of 13.0 feet NGVD). Given no predicted L-29 Section 3 Levee overtopping for this conservative assumed combination of events and recognition that CEPP inflows to WCA-3B (both within the Blue Shanty flow-way and eastern WCA-3B) will utilize controllable structures that may be closed in anticipation of extreme rainfall events, the EN-W preliminary assessment of the WCA-3B design criteria concluded that the proposed CEPP water levels of Alternative 4R2 would not adversely affect the flood control capability of the unmodified eastern segment of the L-29 Levee (or other perimeter levees, which have higher design elevations) bordering WCA-3B. The USACE currently anticipates revisiting the WCA-3B SPF stage during PED, pending final authorization of the CEPP and the establishment of operating criteria for WCA-3B water management structures for a System Operating Manual revision for CEPP implementation.

Maximum stages within the WCA-3B Blue Shanty flow-way and maximum head differential across the L-67D Levee are utilized for the hydraulic, geotechnical, and civil design of the L-67D

Levee for the CEPP TSP Alternative 4R2. Stage duration curves within the interior of the Blue Shanty flow-way, external to the flow-way at the Shark 1 gage in WCA-3B, and within the L-29 Canal (Alternatives 4, 4R, and 4R2 are the only alternatives which include this flow-way component), both west of the CEPP-proposed S-355W L-29 divide structure (within the flow-way, following CEPP removal of this section of the L-29 Levee) and east of the S-355W structure, are shown in Figure 69 (Alternative 4), Figure 70 (Alternative 4R), and Figures 71-72 (Alternative 4R2). The head differential across the L-67D Levee for Alternatives 4 and 4R are shown in Figures 73 through 74 and Figure 76 in both time series format and frequency curve format. The head differential across the L-67D Levee for the CEPP TSP Alternative 4R2 is shown in Figures 75 through 77 in both time series format and frequency curve format; the maximum head differential across the CEPP-proposed L-67D Levee is approximately 1.50 feet during the 1965-2005 RSM-GL period of simulation.

For additional reference, the L-29 Canal stage duration curves for the CEPP ECB, CEPP FWO, and CEPP alternatives 1 through 4R2 are shown in Figure 78 through 81 (stages correspond to the western reach of the L-29 Canal for Alternatives 4, 4R, and 4R2, west of the S-355W structure); peak stages are indicated on Figure 79 and Figure 81. Peak L-29 Canal stages for CEPP will need to be considered for future implementation of the DOI TTNS roadway modifications, including the potential need to further raise the eastern portion of the Tamiami Trail roadway, east of the CEPP-proposed S-355W L-29 divide structure. Peak simulated L-29 Canal stages for Alternative 4R2 are 9.59 feet NGVD west of the divide structure and 9.50 feet NGVD east of the divide structure (refer to Figures 71 and 72).

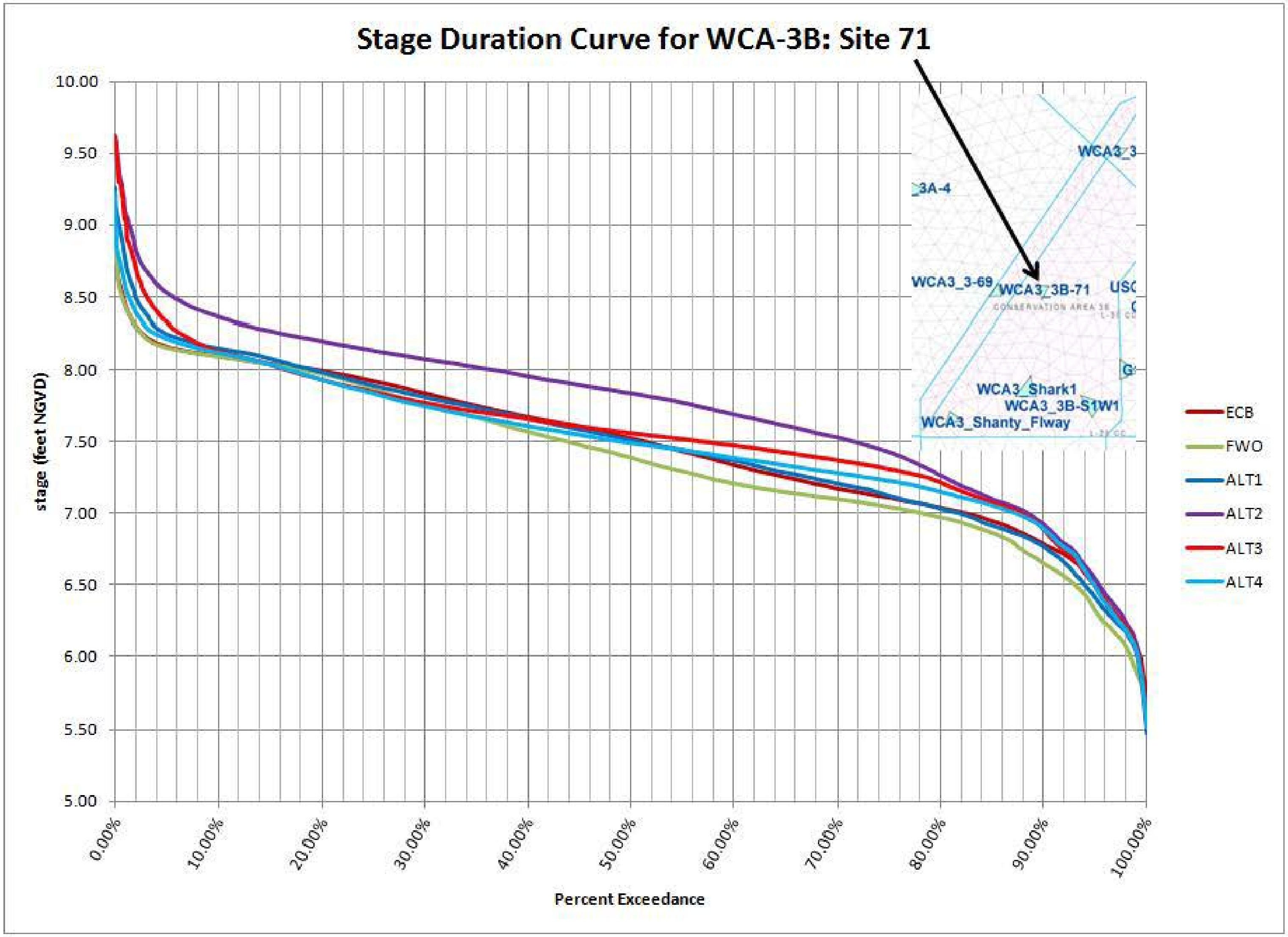


Figure 58: WCA-3B Site 71 stage duration curves for CEPP baselines and CEPP Alternatives 1 through 4

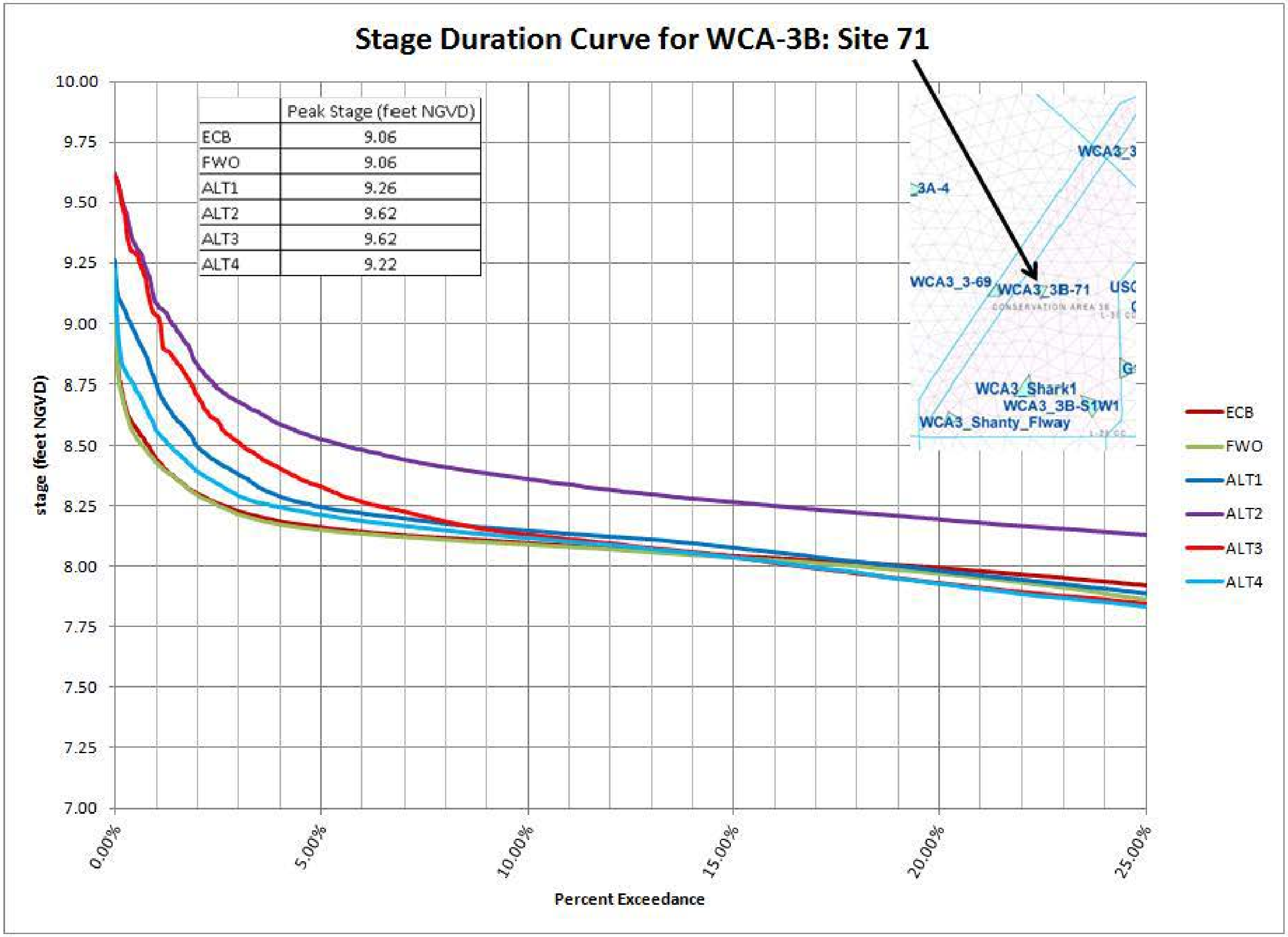


Figure 59: WCA-3B Site 71 stage duration curves for CEPP baselines and CEPP Alternatives 1 through 4 (Upper 25%)

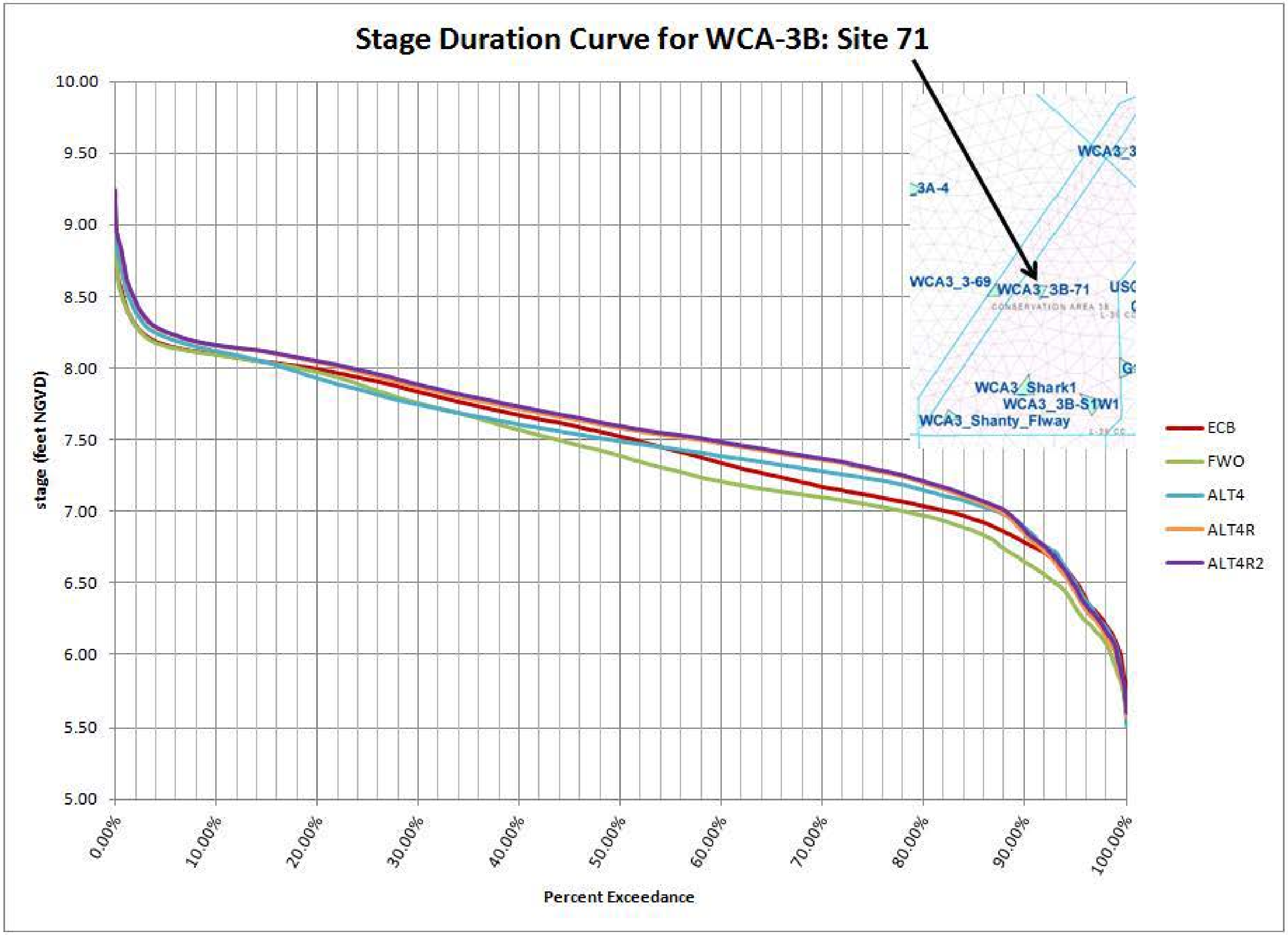


Figure 60: WCA-3B Site 71 stage duration curves for CEPP baselines and CEPP Alternatives 4,4R, and 4R2

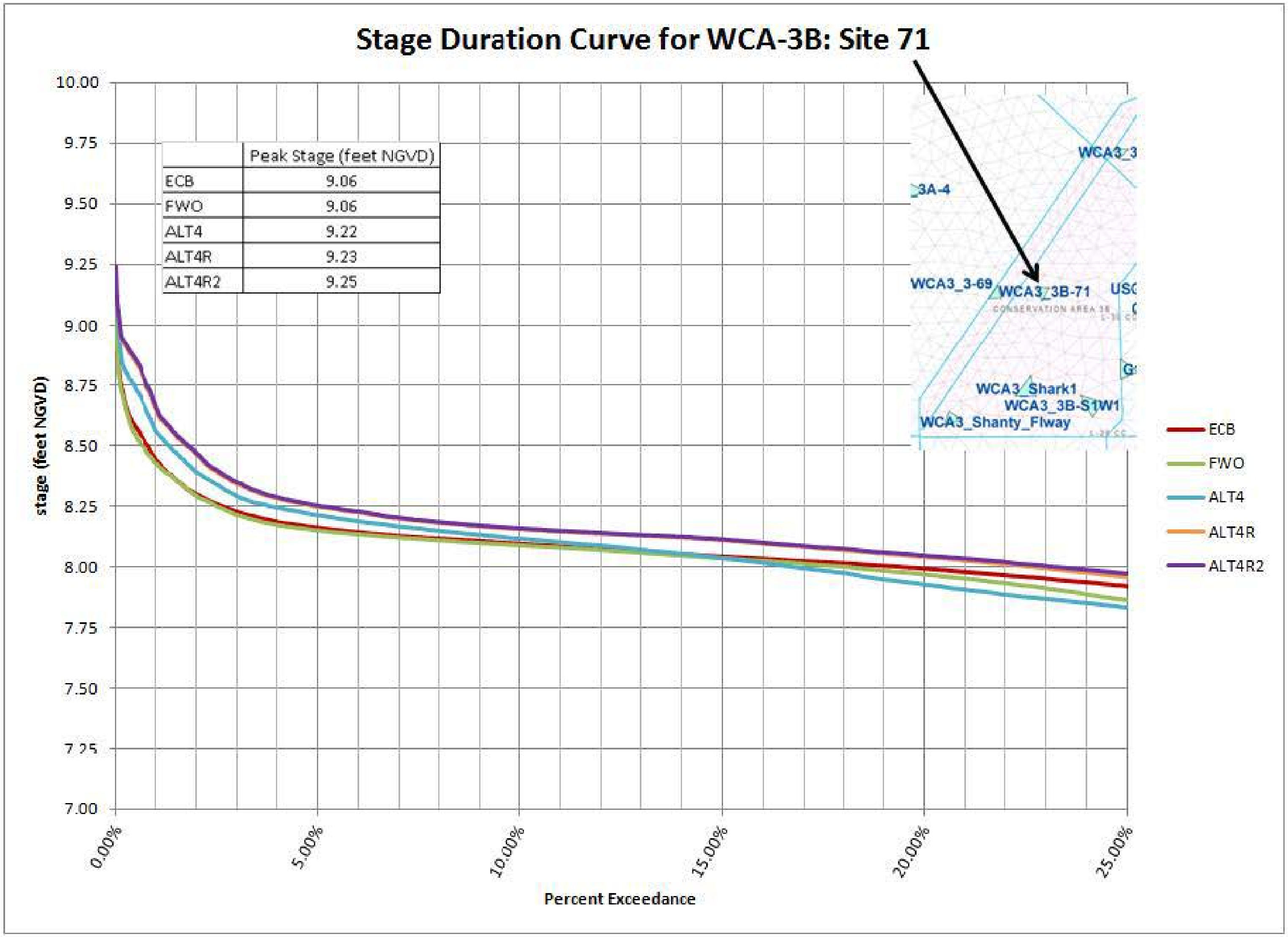


Figure 61: WCA-3B Site 71 stage duration curves for CEPP baselines and CEPP Alternatives 4, 4R, and 4R2 (upper 25%)

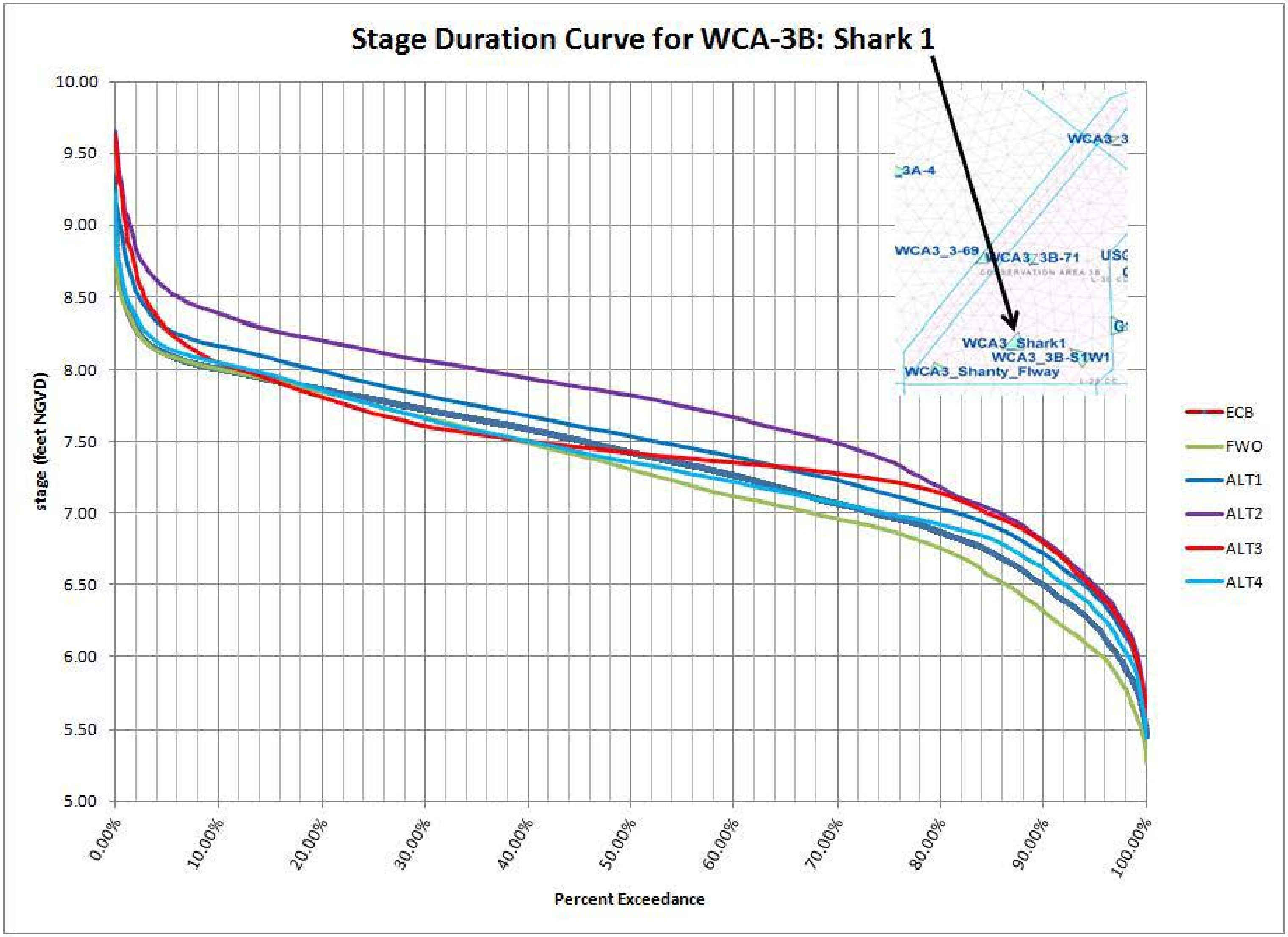


Figure 62: WCA-3B Shark-1 stage duration curves for CEPP baselines and CEPP Alternatives 1 through 4

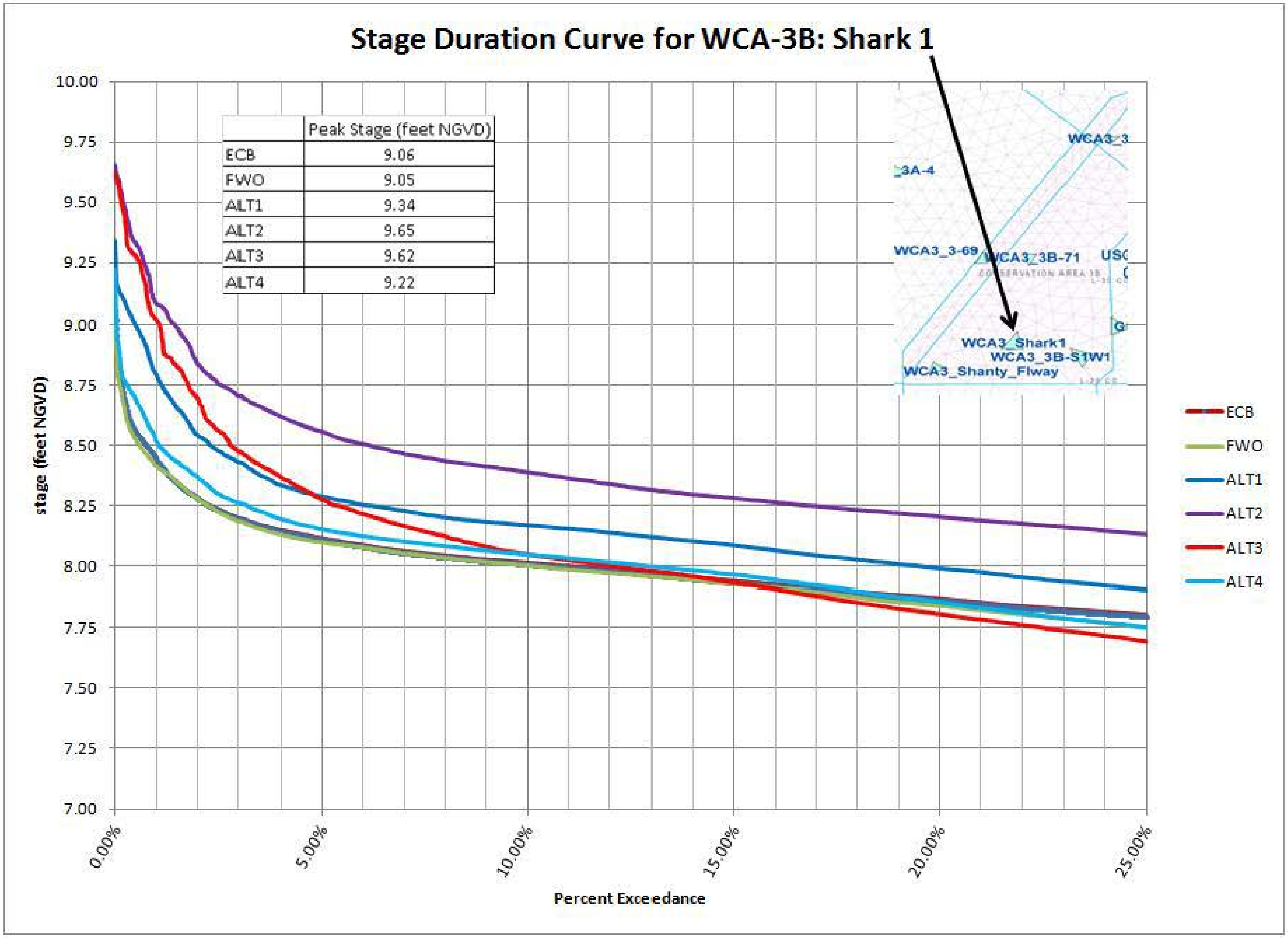


Figure 63: WCA-3B Shark-1 stage duration curves for CEPP baselines and CEPP Alternatives 1 through 4 (Upper 25%)

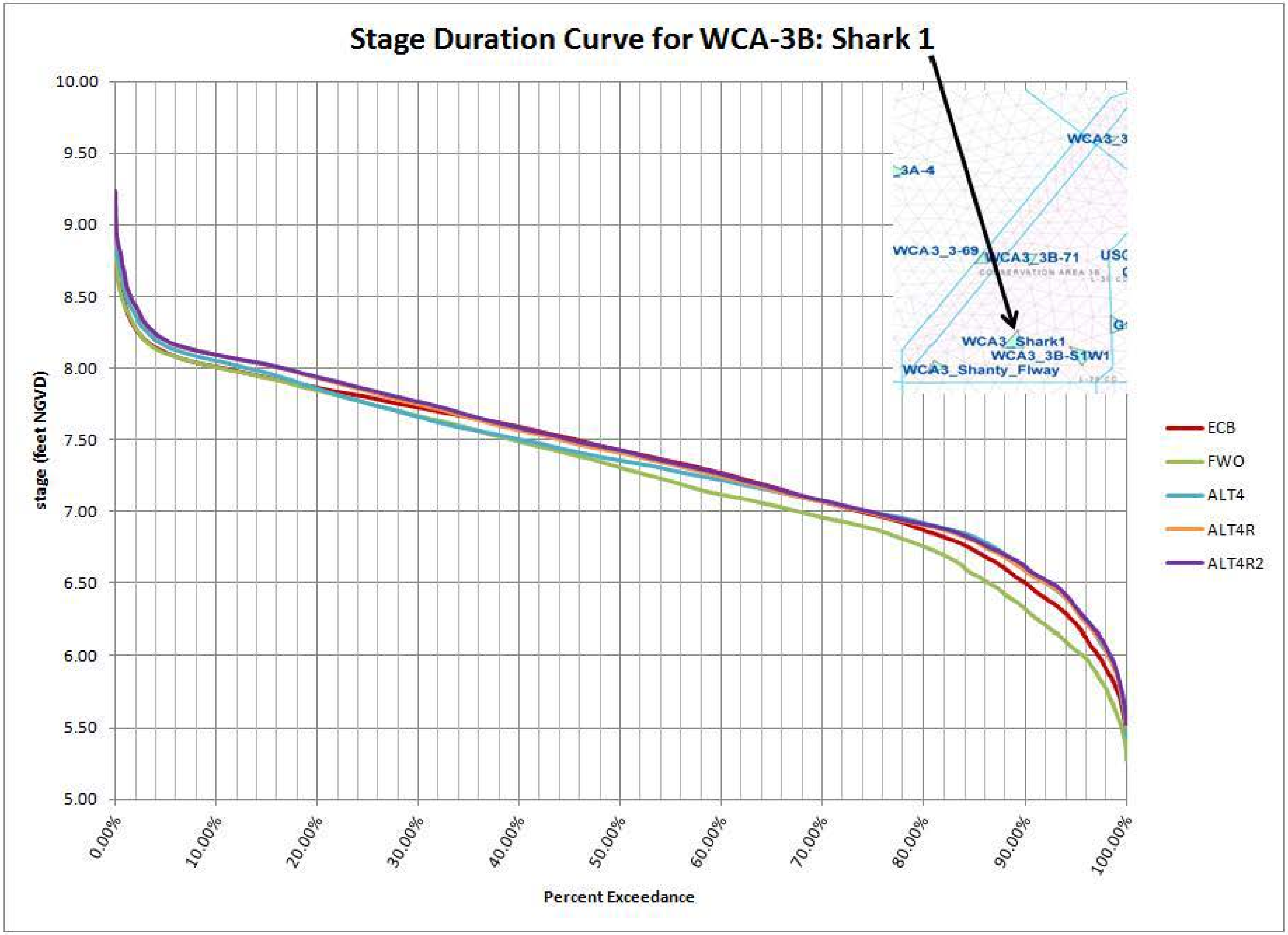


Figure 64: WCA-3B Shark-1 stage duration curves for CEPP baselines and CEPP Alternatives 4, 4R, and 4R2

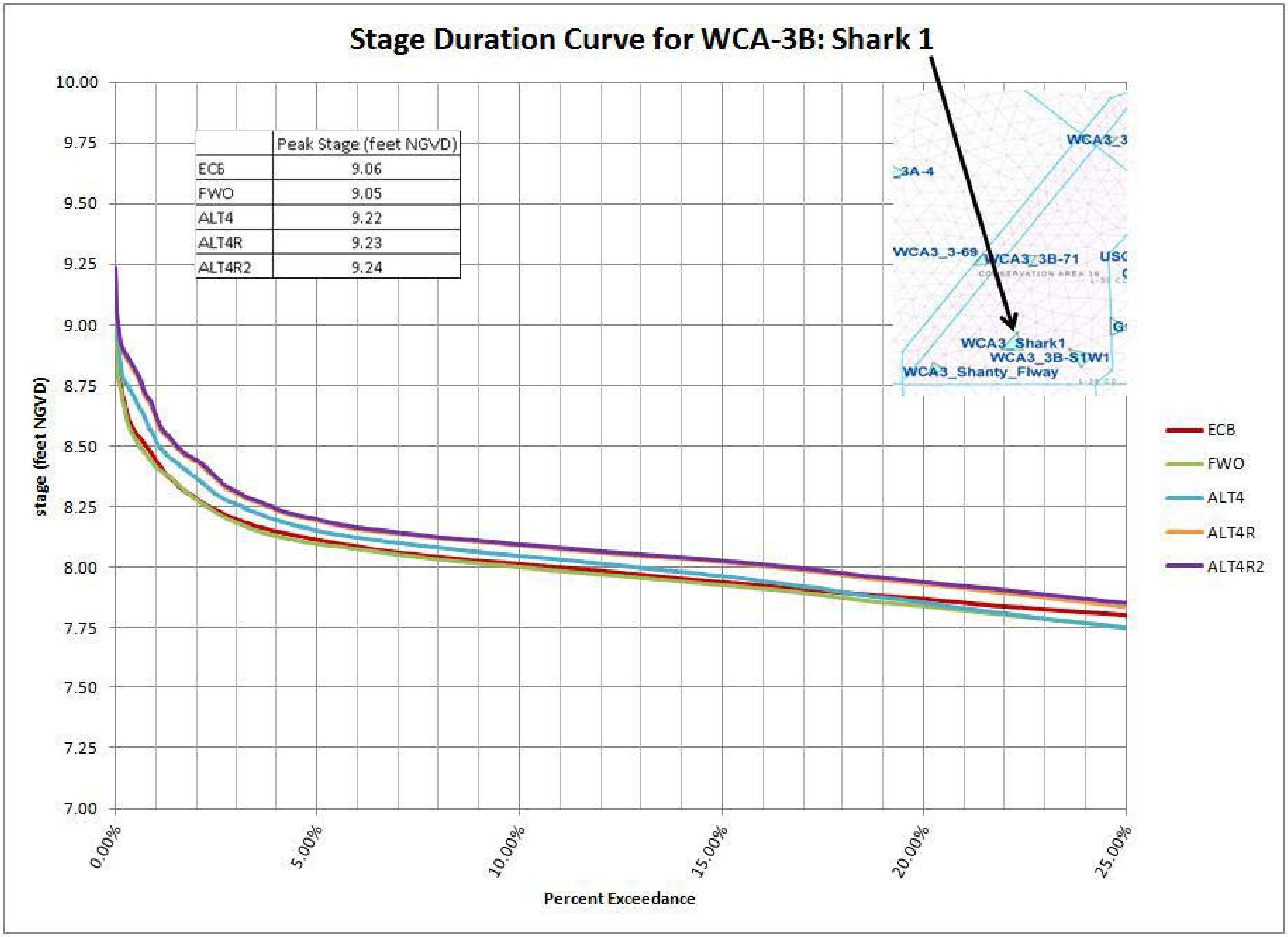


Figure 65: WCA-3B Shark-1 stage duration curves for CEPP baselines and CEPP Alternatives 4, 4R, and 4R2 (upper 25%)

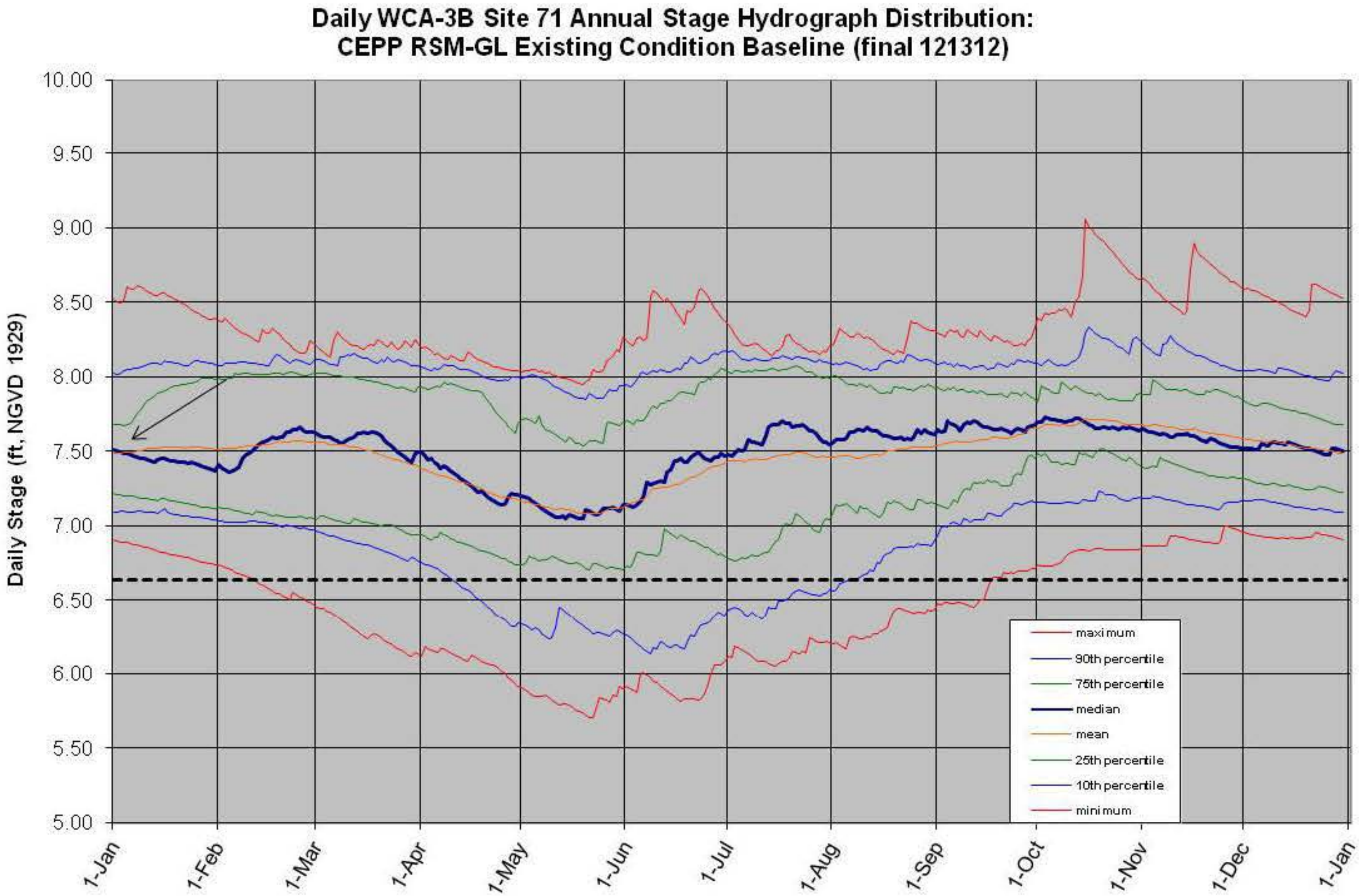


Figure 66: WCA-3B Site 71 annual average stage hydrographs for CEPP ECB Baseline

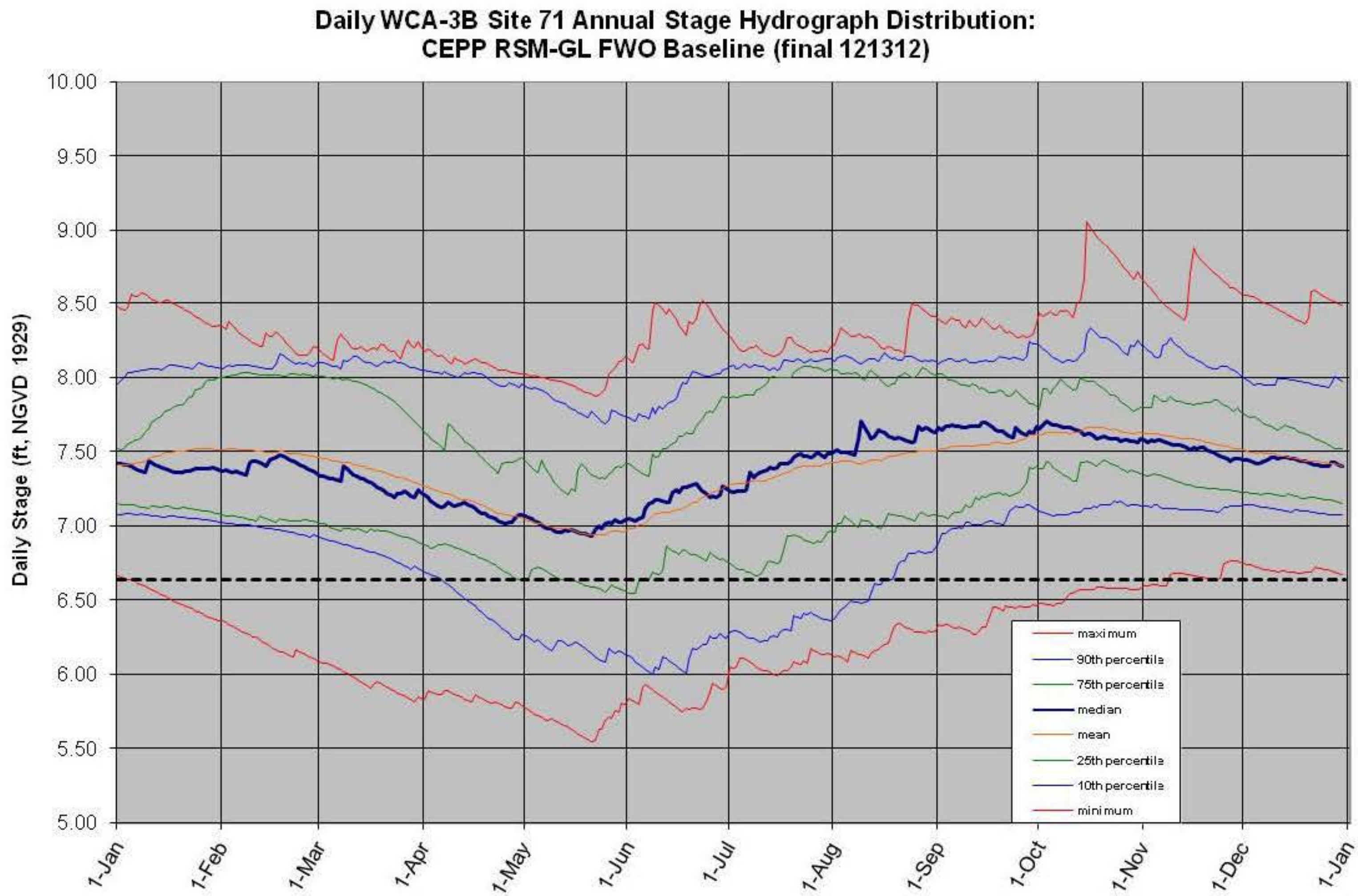


Figure 67: WCA-3B Site 71 annual average stage hydrographs for CEPP FWO Baseline

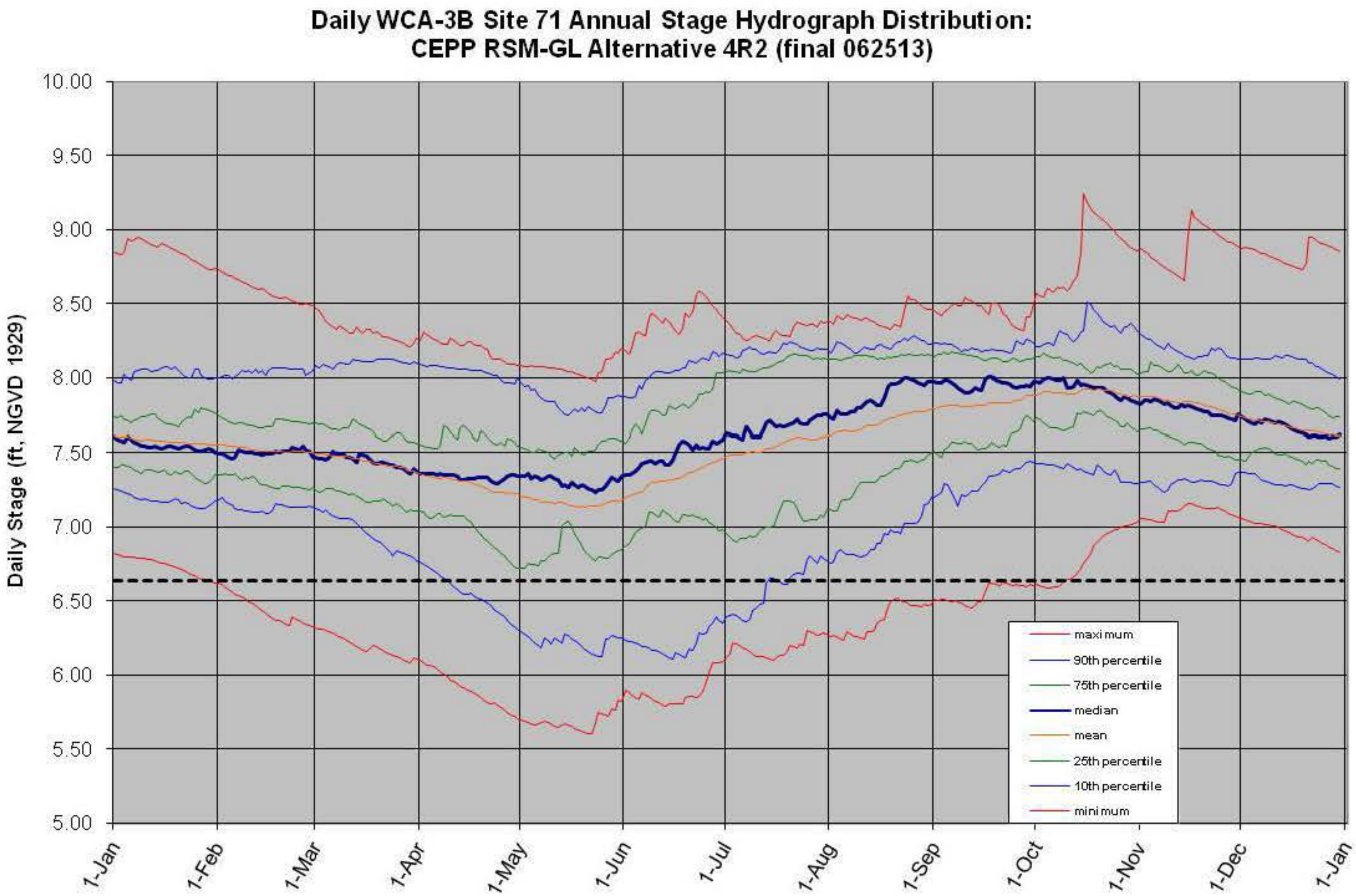


Figure 68: WCA-3B Site 71 annual average stage hydrographs for CEPP Alternative 4R2

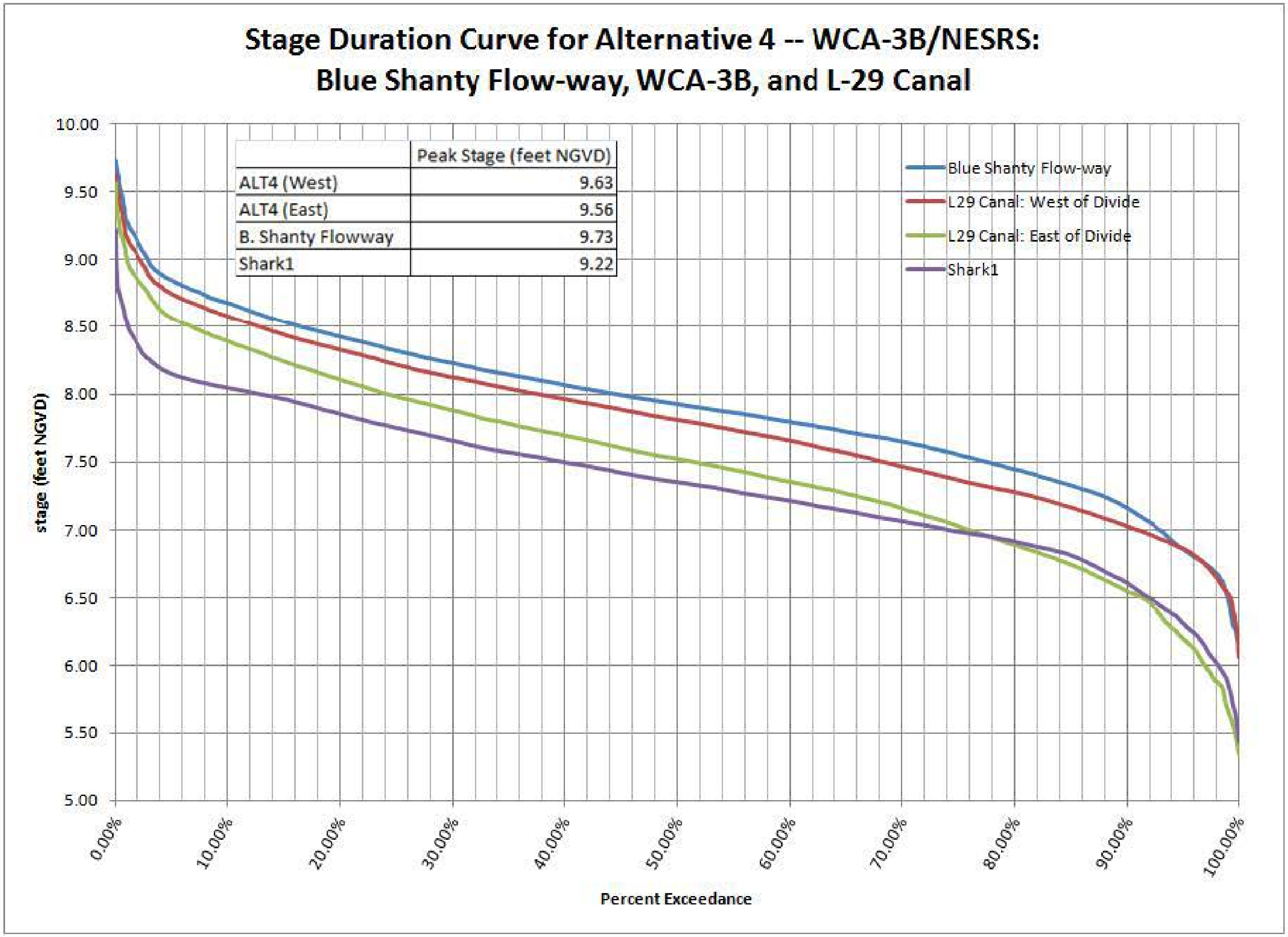
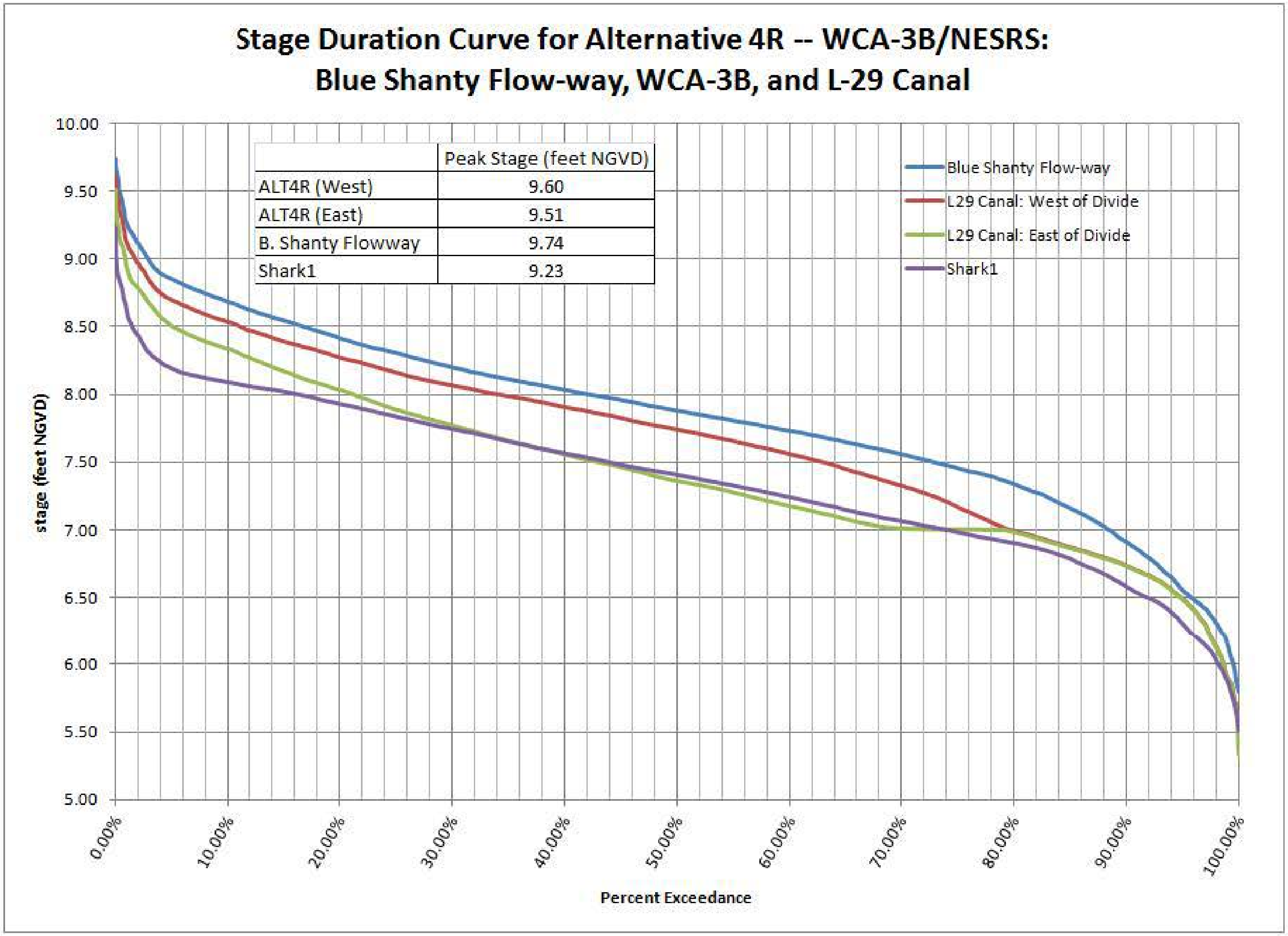
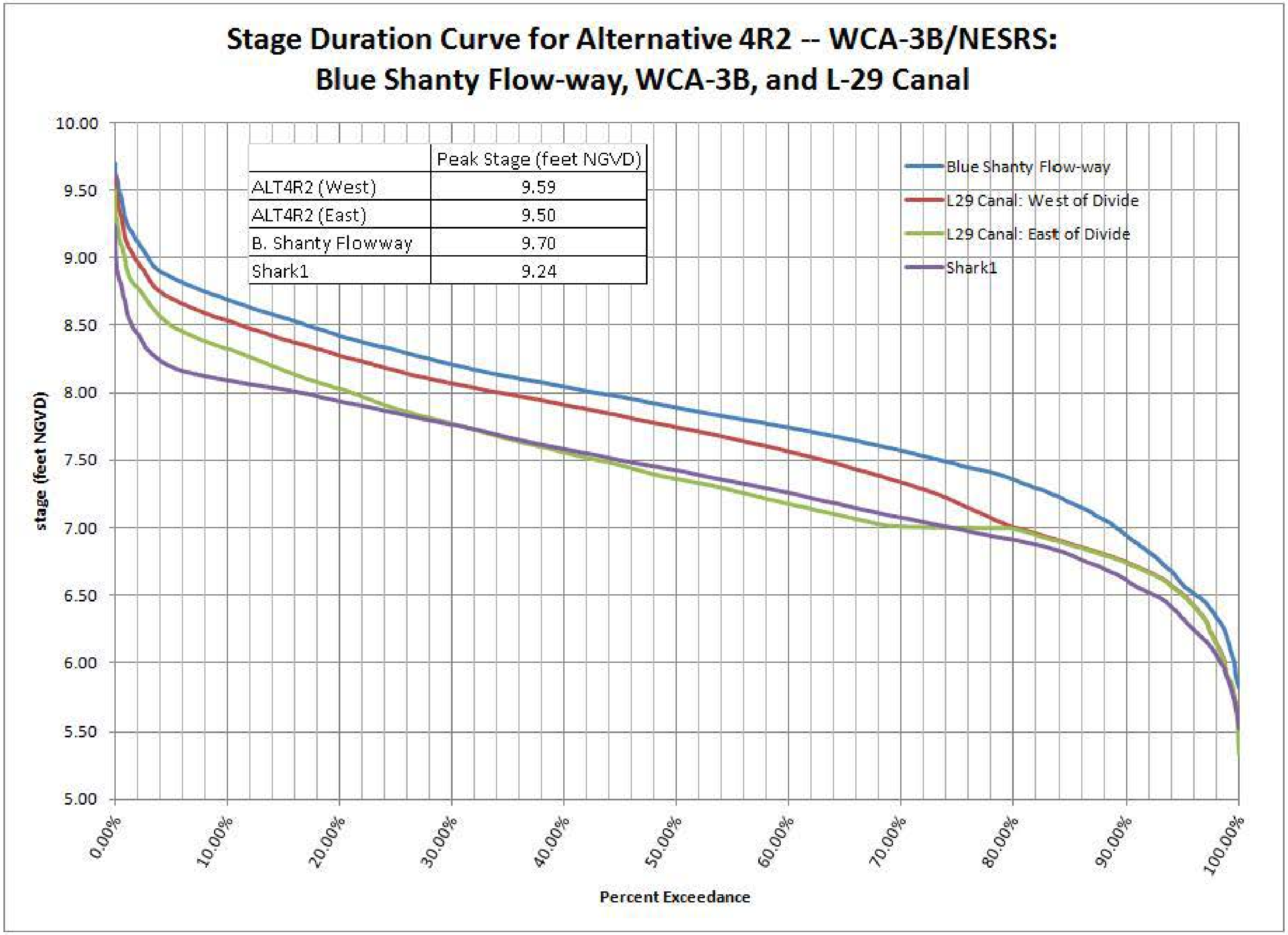


Figure 69: L-29 Canal, WCA-3B, and Blue Shanty Flow-way stage duration curves for CEPP Alternative 4



	Peak Stage (feet NGVD)
ALT4R (West)	9.60
ALT4R (East)	9.51
B. Shanty Flowway	9.74
Shark1	9.23

Figure 70: L-29 Canal, WCA-3B, and Blue Shanty Flow-way stage duration curves for CEPP Alternative 4R



	Peak Stage (feet NGVD)
ALT4R2 (West)	9.59
ALT4R2 (East)	9.50
B. Shanty Flowway	9.70
Shark1	9.24

Figure 71: L-29 Canal, WCA-3B, and Blue Shanty Flow-way stage duration curves for CEPP Alternative 4R2

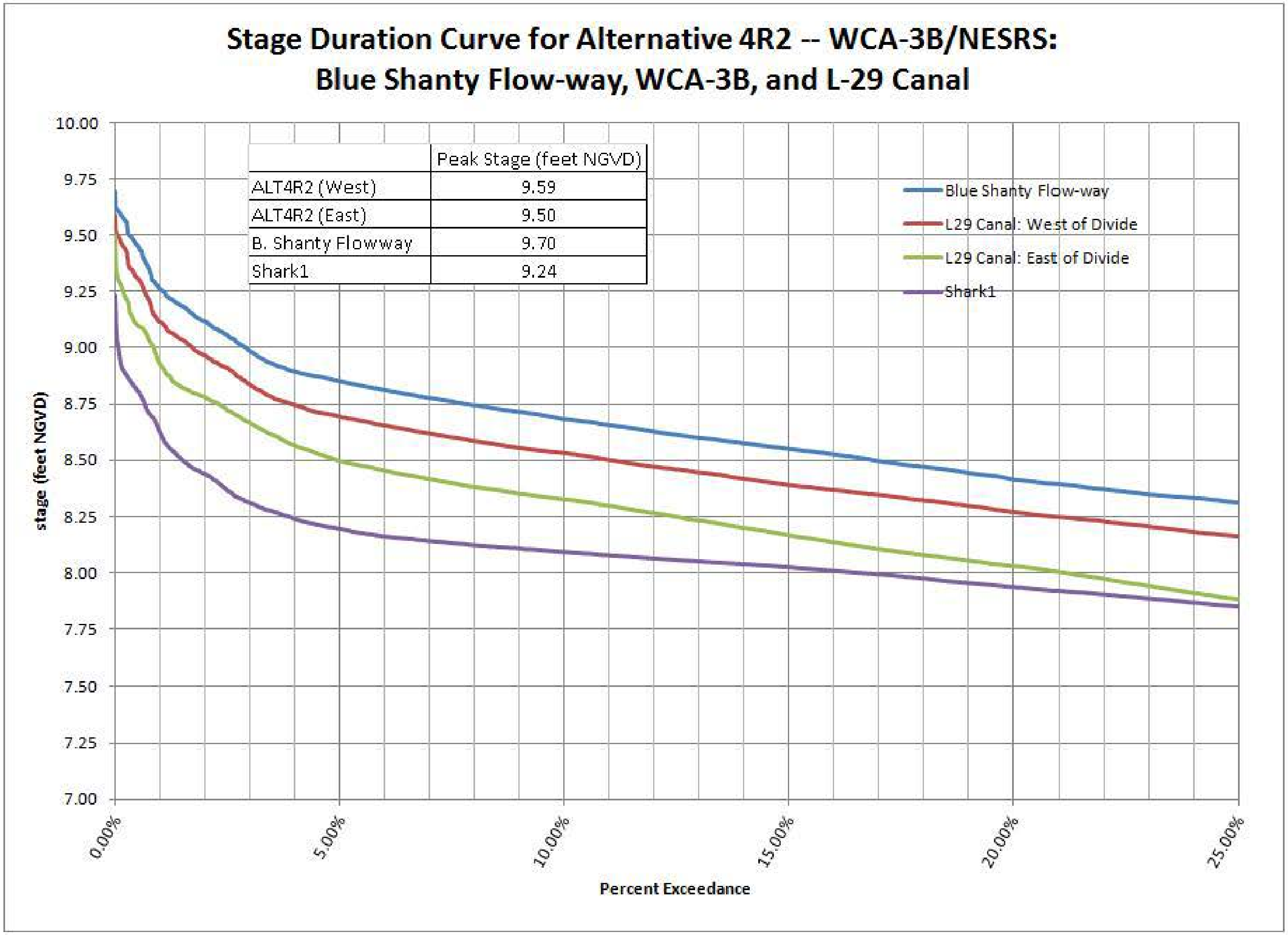


Figure 72: L-29 Canal, WCA-3B, and Blue Shanty Flow-way stage duration curves for CEPP Alternative 4R2 (Upper 25%)

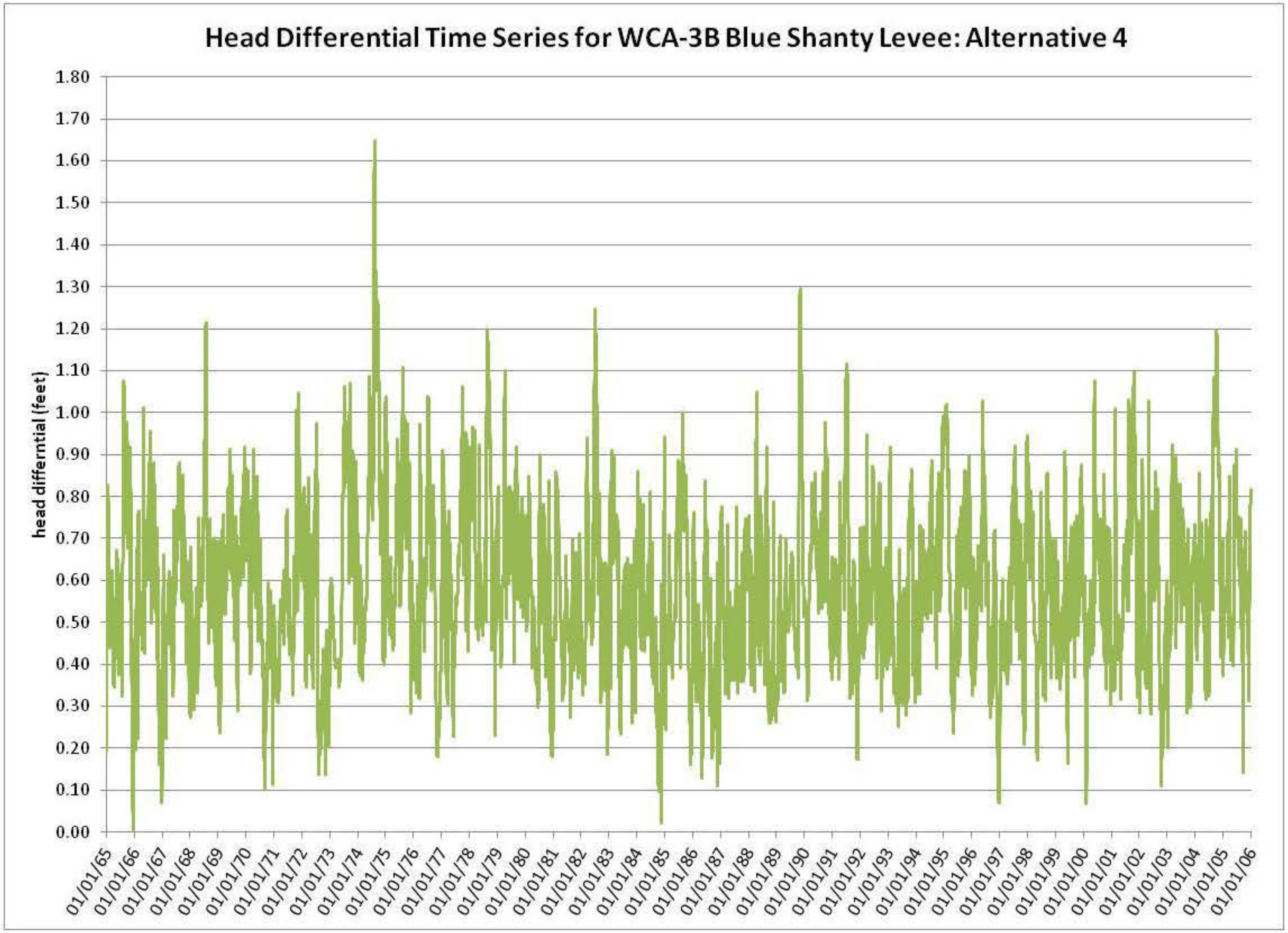


Figure 73: L-67D head differential time series for CEPP Alternative 4

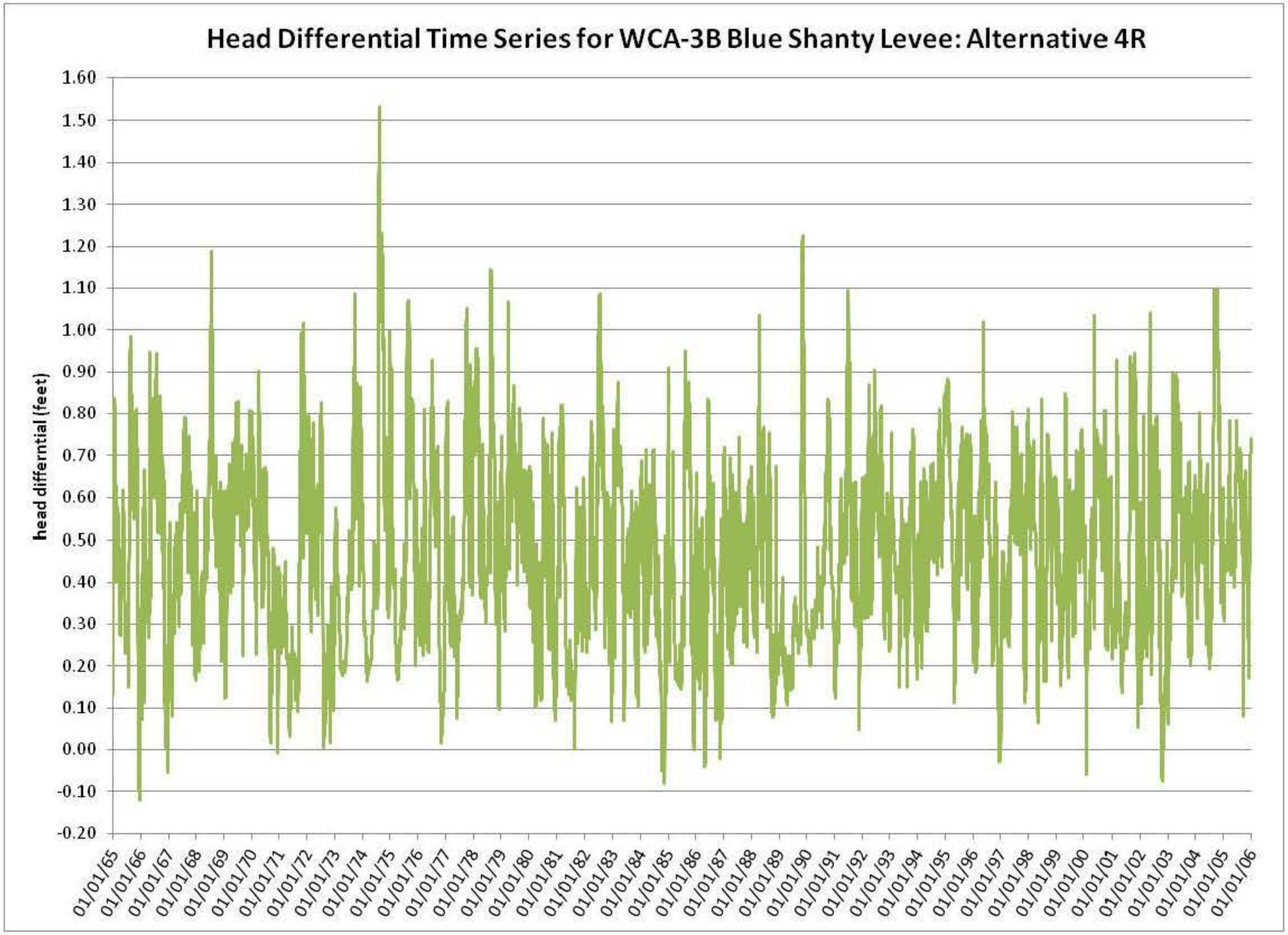


Figure 74: L-67D head differential time series for CEPP Alternative 4R

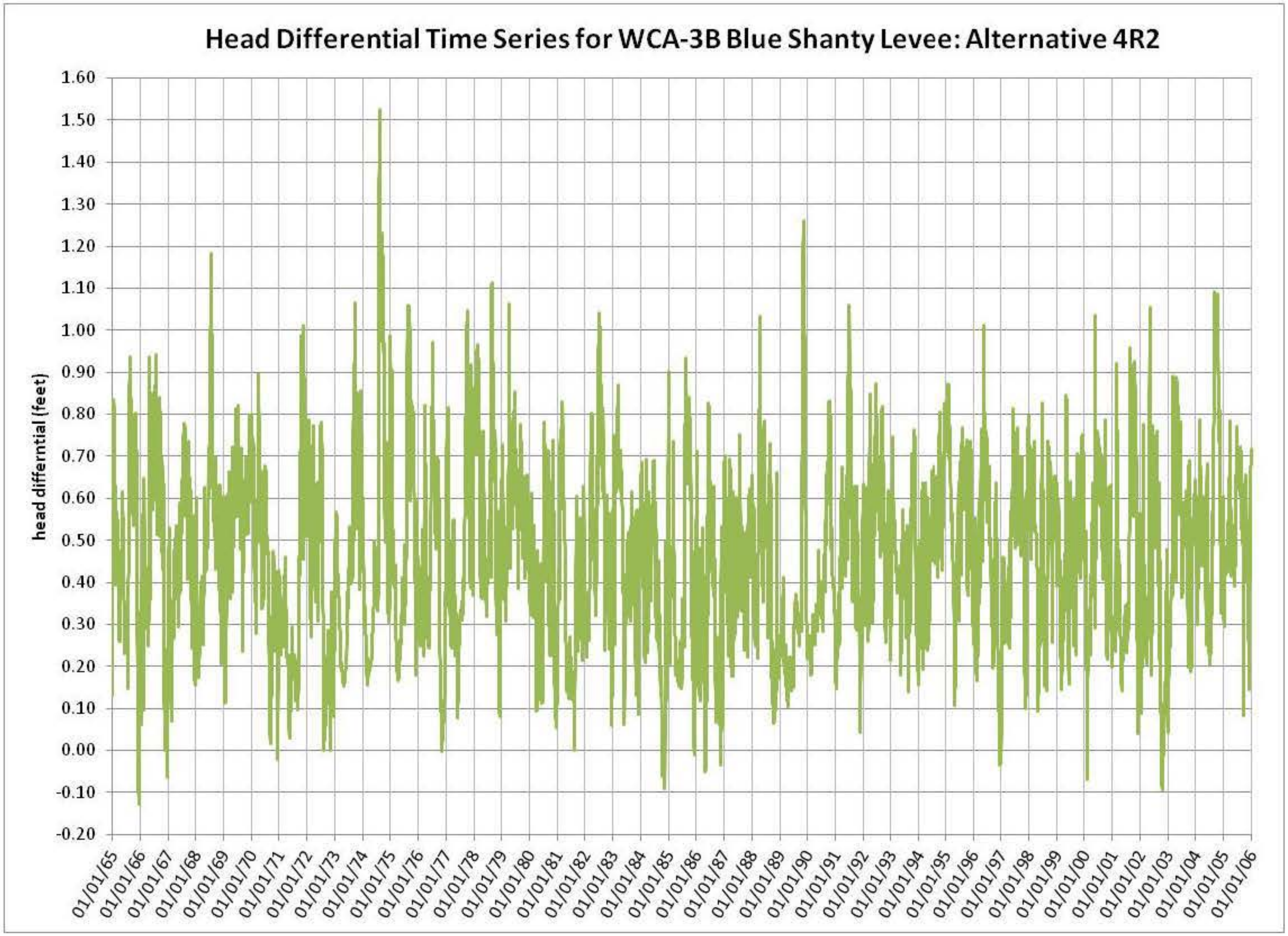


Figure 75: L-67D head differential time series for CEPP Alternative 4R2

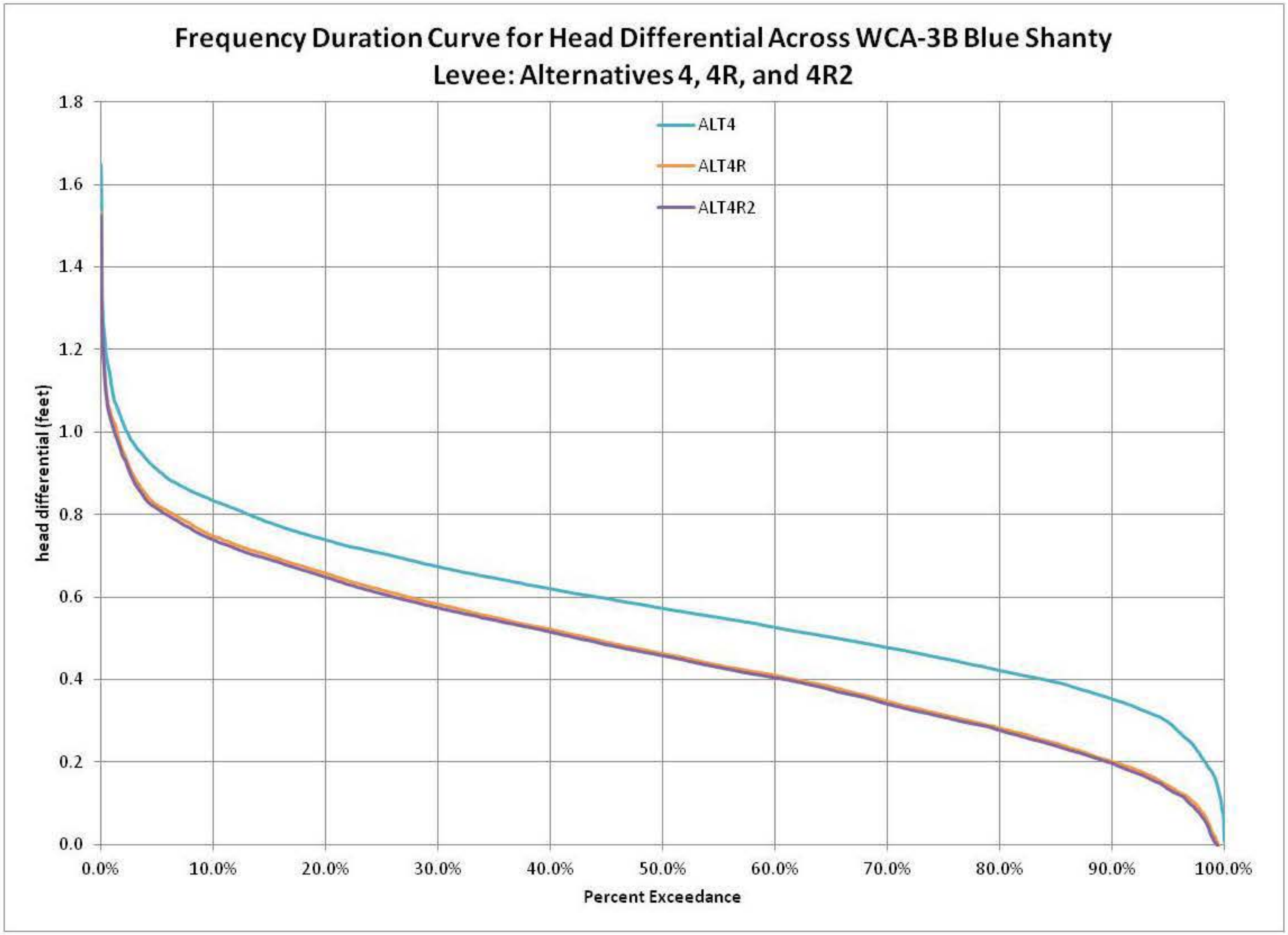


Figure 76: L-67D head differential frequency curve for CEPP Alternatives 4, 4R, and 4R2

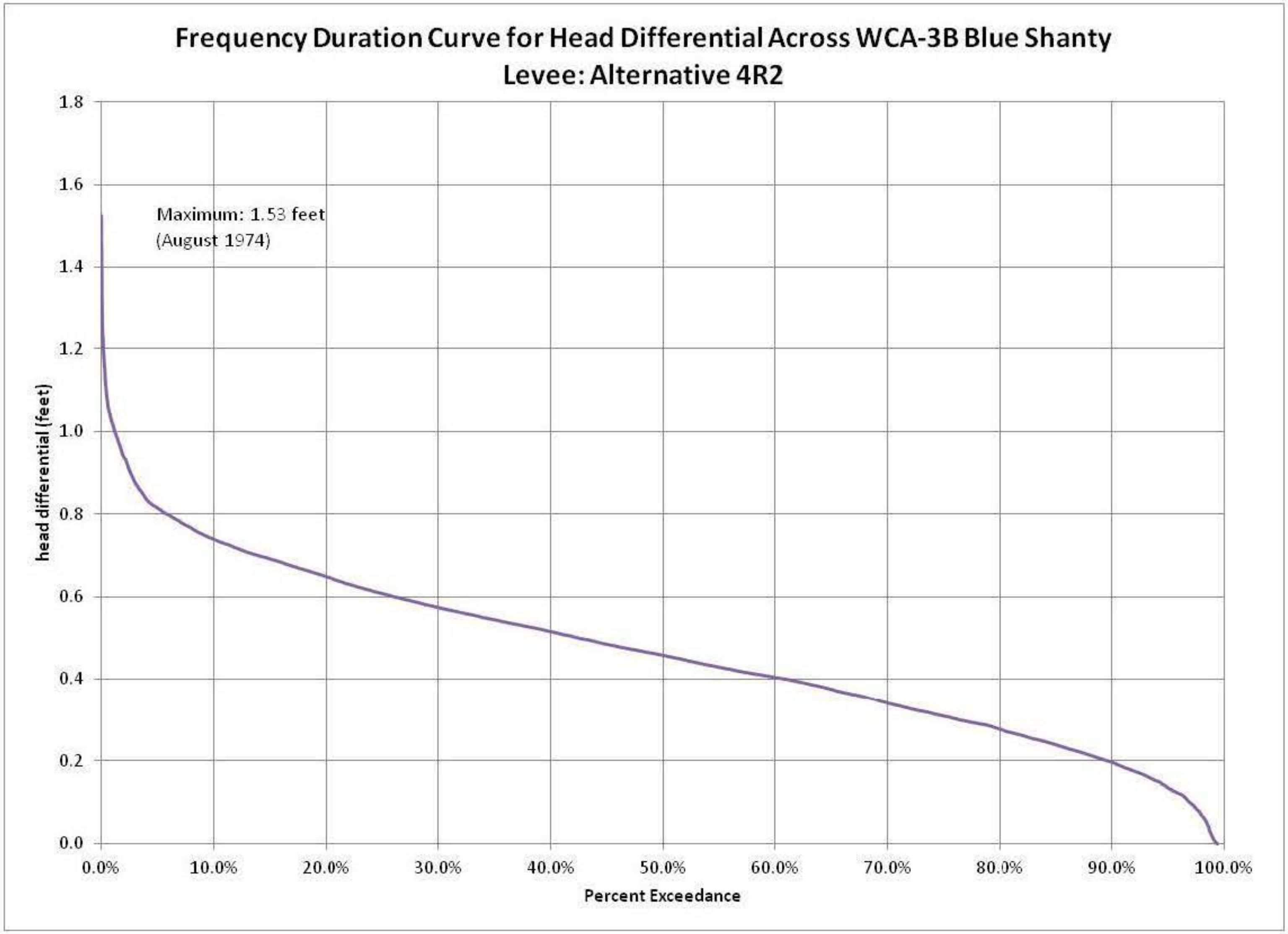


Figure 77: L-67D head differential frequency curve for CEPP Alternatives 4R2

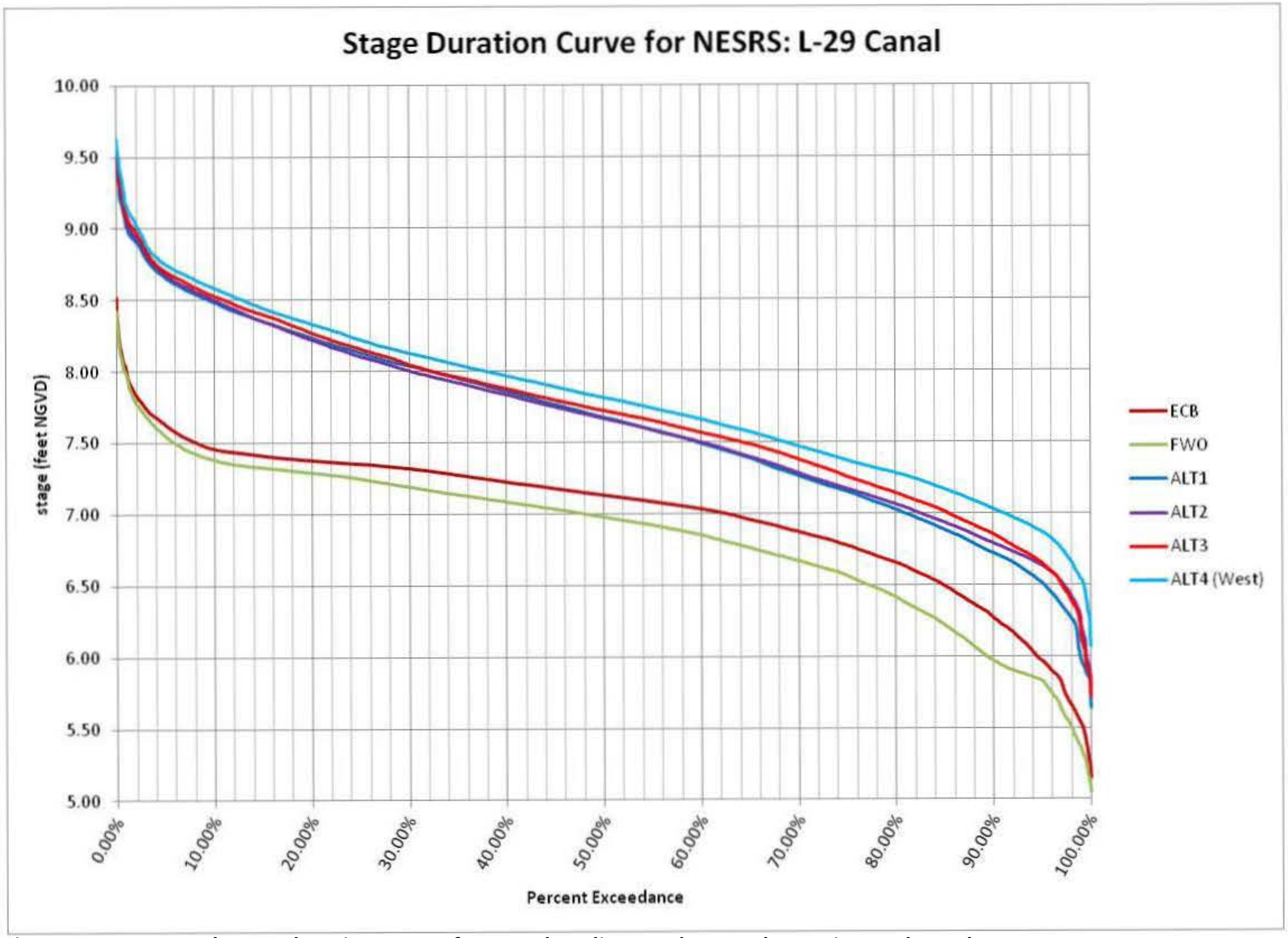


Figure 78: L-29 Canal stage duration curves for CEPP baselines and CEPP Alternatives 1 through 4

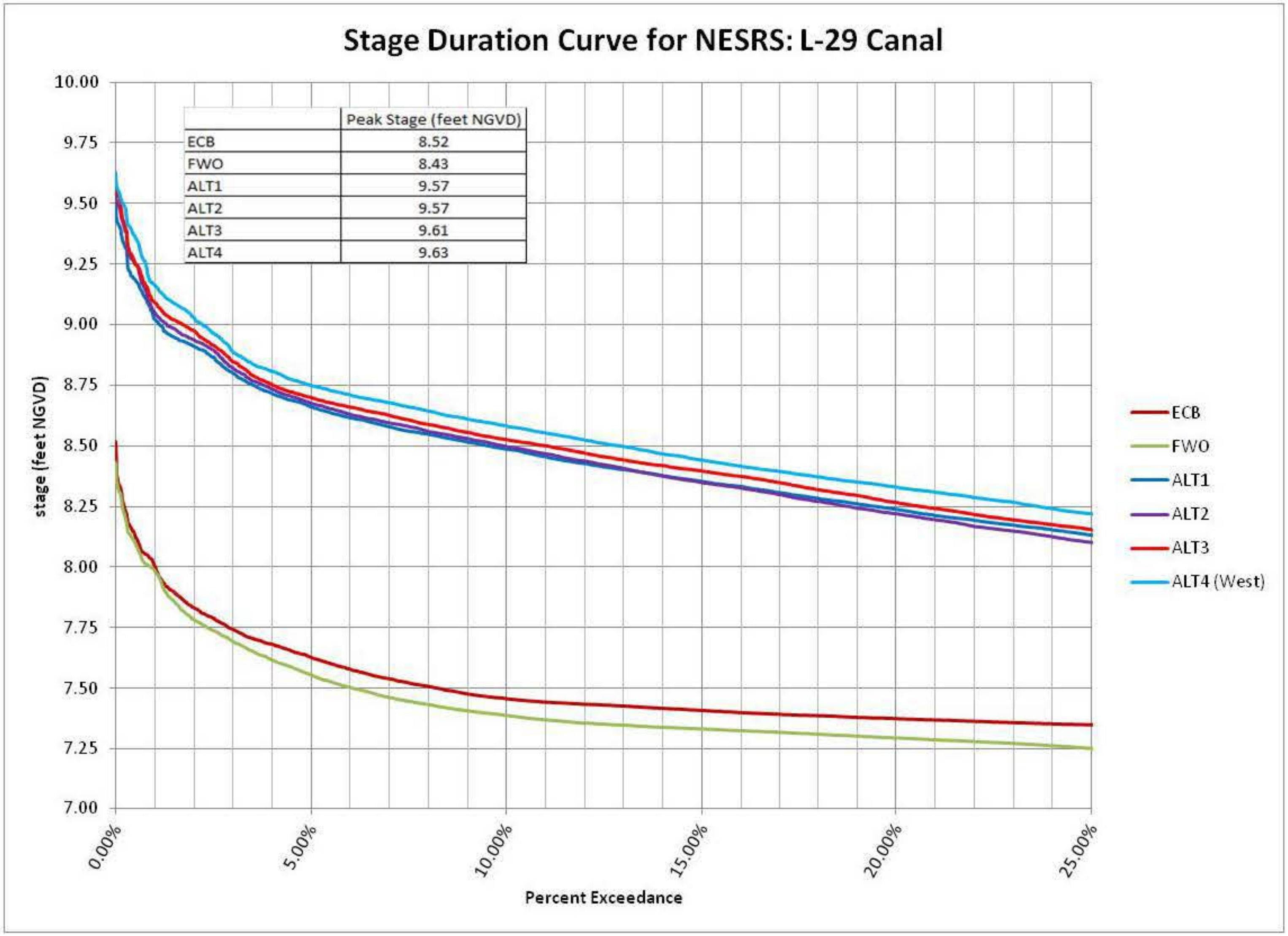


Figure 79: L-29 Canal stage duration curves for CEPP baselines and CEPP Alternatives 1 through 4 (Upper 25%)

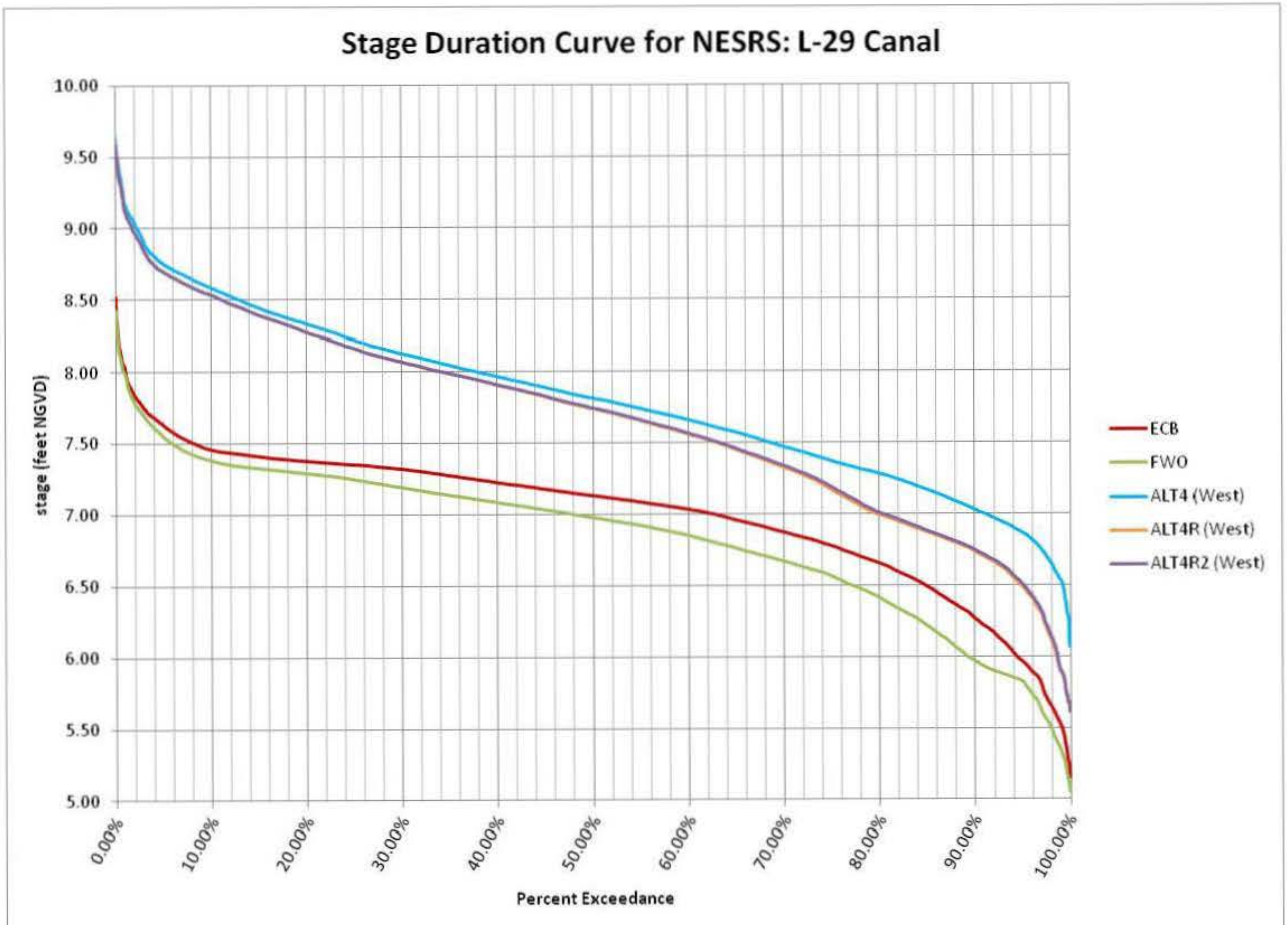
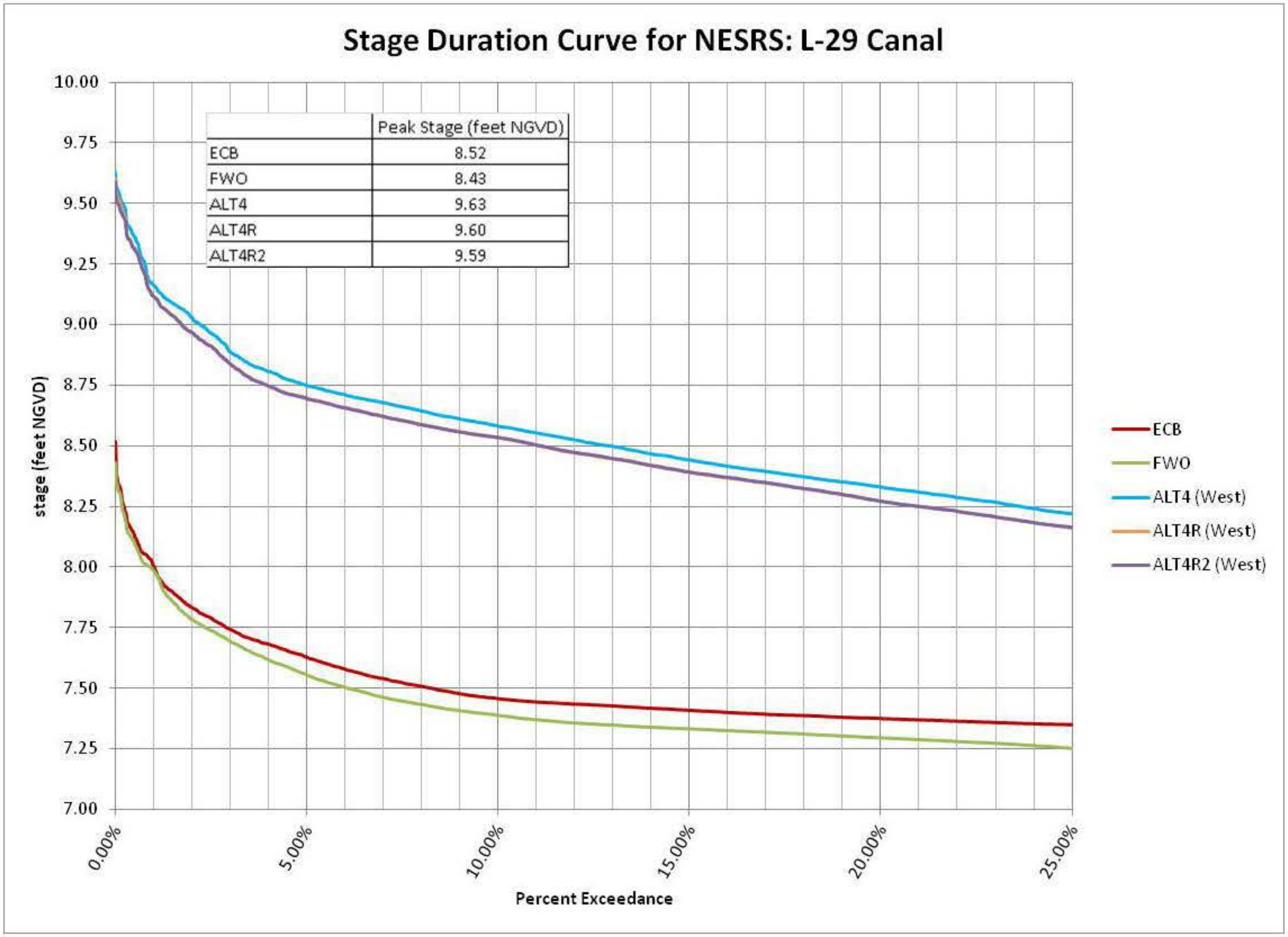


Figure 80: L-29 Canal stage duration curves for CEPP baselines and CEPP Alternatives 4, 4R, and 4R2



	Peak Stage (feet NGVD)
ECB	8.52
FWO	8.43
ALT4	9.63
ALT4R	9.60
ALT4R2	9.59

Figure 81: L-29 Canal stage duration curves for CEPP baselines and CEPP Alternatives 4, 4R, and 4R2 (upper 25%)

3.2.3. Lake Okeechobee Herbert Hoover Dike Design Considerations

Herbert Hoover Dike (HHD) design considerations, with particular consideration of the effects of operational flexibility within the 2008 Lake Okeechobee Regulation Schedule, are addressed in Section A.8.3.2.3 of the Engineering Appendix. Information is presented in the Engineering Appendix for CEPP alternatives 1 through 4R2.

3.2.4. 8.5 Square Mile Area Flood Mitigation Performance

The 8.5 Square Mile Area (8.5 SMA) is a primarily residential area adjacent to, but west of, the L-31N Canal. The 8.5 SMA, which is also known as the Las Palmas community, is bordered on both the west and north by NESRS (Figure 82). The community has water management infrastructure consisting of a perimeter levee, a seepage collection canal, a pump station (S-357), and a southern detention cell meant to collectively provide flood mitigation as part of the MWD Project.

Stages within the 8.5 SMA, located along the eastern boundary of ENP, do not change significantly between the CEPP ECB and the FWO. The 8.5 SMA project components and operations are unchanged between the ECB and FWO modeling assumptions, with each baseline condition assuming operations of S-357 and S-331 as defined in the 2011 8.5 SMA Interim Operational Criteria; the S-357 pump station is limited to a 125 cfs average daily discharge rate, and S-331 flood mitigation operations for the 8.5 SMA are triggered based on the stage at the LPG-2 monitoring gage (located within the protected area, along the western perimeter levee).

The CEPP alternatives modify the FWO operations of the S-357 pump station, in an effort to increase discharges from the 8.5 SMA detention cell to the C-111 South Dade North Detention Area and reduce the reliance on the S-331 pump station in L-31N to provide flood mitigation for the 8.5 SMA protected area. The protected portion of the 8.5 SMA is represented by 3 model grid cells in the RSM-GL (Figure 83), and the resolution of the RSM-GL is extremely limiting for adequate representation of the 8.5 SMA project features. Prior to implementation of CEPP, further technical investigations and potentially additional hydrologic/hydraulic modeling with a higher resolution model will likely be needed for the 8.5 SMA operations. The current MWD 8.5 SMA configuration was identified in the USACE C&SF MWD 8.5 SMA General Reevaluation Report (2000 GRR), which provided a detailed quantification of potential affects to 8.5 SMA flood mitigation performance and potential affects to adjacent ENP wetlands supported by ModBranch hydrologic modeling.

RSM-GL final array modeling of Alternatives 1 through 4 indicated that stages within the 8.5 SMA were lowered by approximately 0.25 feet during wet conditions for RSM-GL grid cells 2965 (Figure 84) and 2962 (Figure 85), compared to the FWO; within the resolution of the RSM-GL model, these grid cells represent northern and southeastern 8.5 SMA, respectively. However, of concern with Alternatives 1 through 4, stages within the southwest portion of the 8.5 SMA, represented by RSM-GL grid cell 2749, were increased by approximately 0.3-0.6 feet, compared to the FWO, under all hydrologic conditions (Figure 86). These alternatives maintained increased utilization of the S-357 pump station to provide effective flood mitigation for the 8.5 SMA protected area but did not include lowering of the overflow weirs' elevations within the 8.5 SMA detention area (crest elevations for the S-360W and S-360E weirs were maintained at the elevations specified for the 2011 Interim Operations Plan for 8.5 SMA, corresponding to overflow depths of 4.0 and 3.5 feet, respectively); consistent with previous field observations during S-357 interim operations, the CEPP modeling demonstrated that increased operational depths within the 8.5 SMA detention area may potentially cause increased groundwater stages within the southwestern portion of the 8.5 SMA protected area.

The 8.5 SMA detention cell weirs were lowered with Alternative 4R and Alternative 4R2 to allow overflow when depths exceeded 1.0 feet, which resulted in performance improvements within the southwestern portion of the 8.5 SMA protected area, RSM-GL grid cell 2749. RSM-GL modeling of Alternative 4R and Alternative 4R2 indicates that stages within the 8.5 SMA are lowered by approximately 0.25-0.50 feet during wet conditions for the three RSM-GL grid cells that represent the protected portion of the 8.5 SMA, compared to the FWO (Figures 84 through 86).

During the PED phase of CEPP, further technical investigations and potentially additional hydrologic/hydraulic modeling with a higher resolution model will likely be needed for the 8.5 SMA operations.

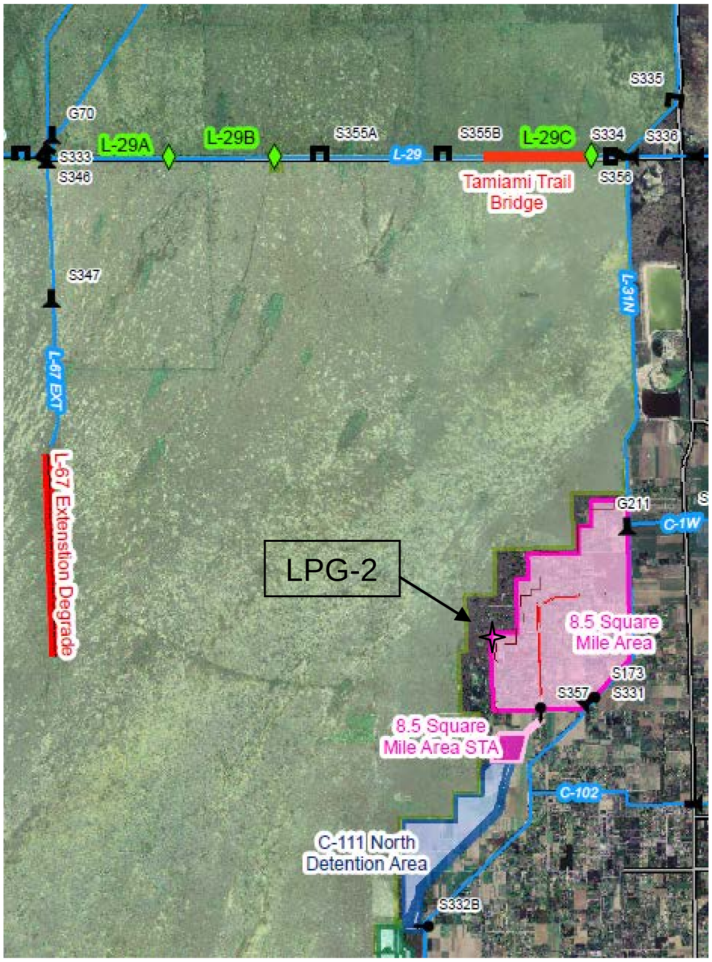


Figure 82: Location map for 8.5 SMA

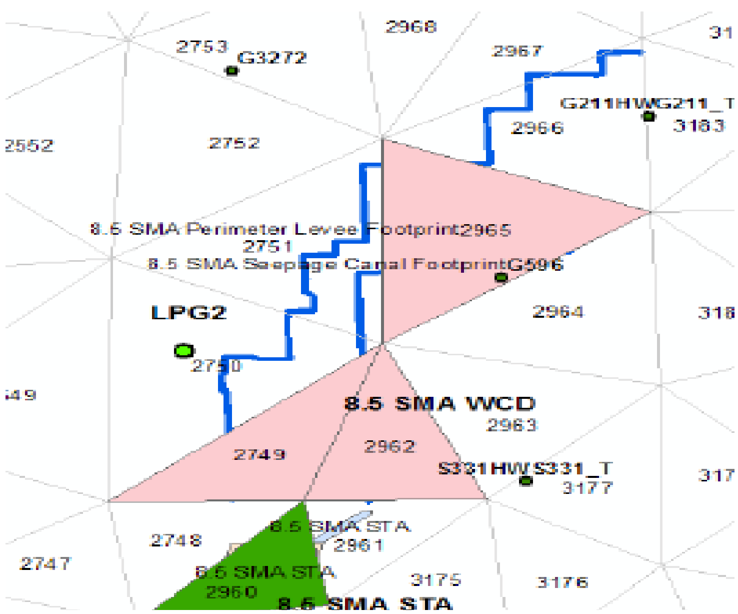


Figure 83: RSM-GL grid cell representation of the 8.5 SMA

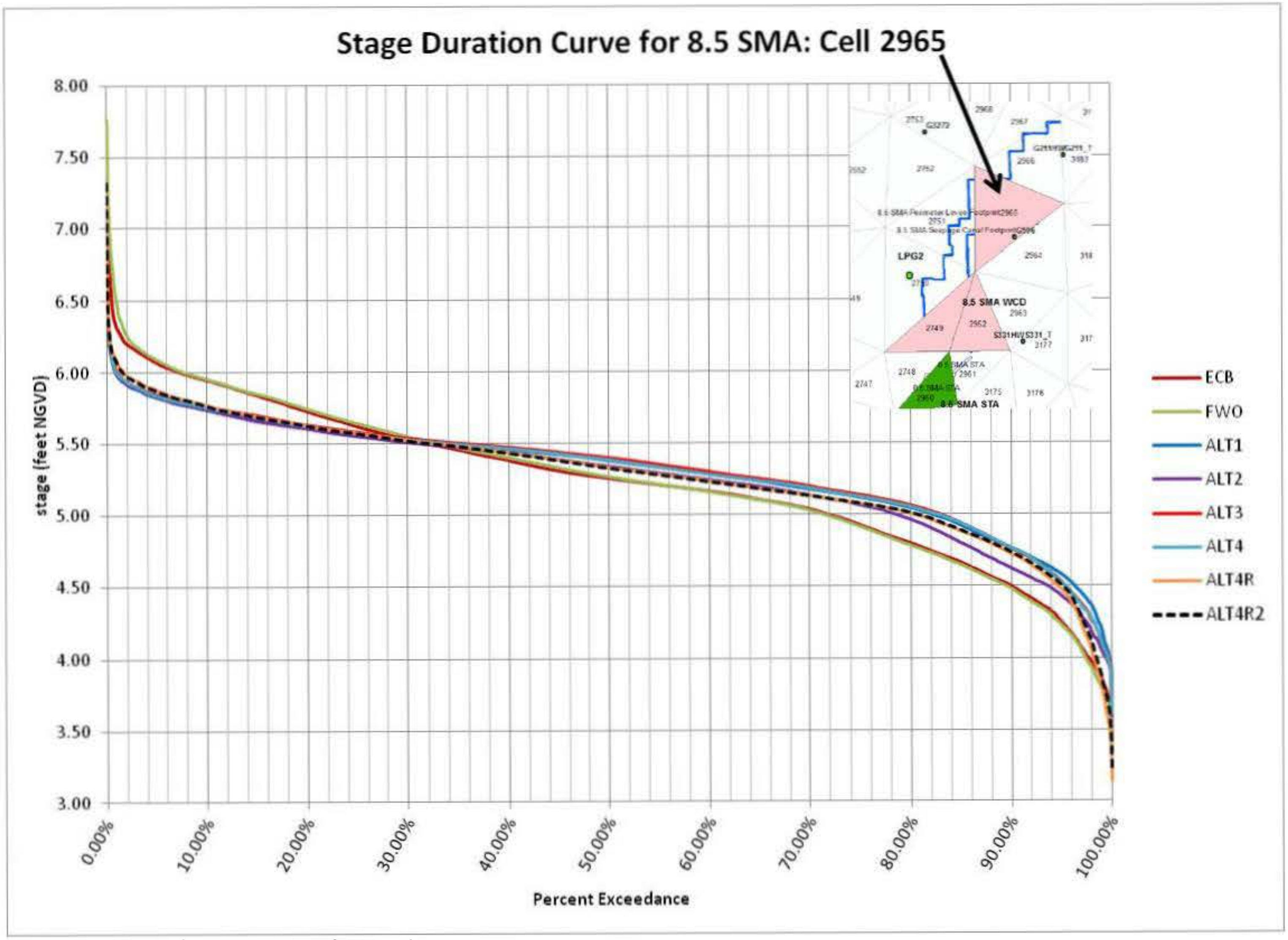


Figure 84: Stage duration curve for north 8.5 SMA

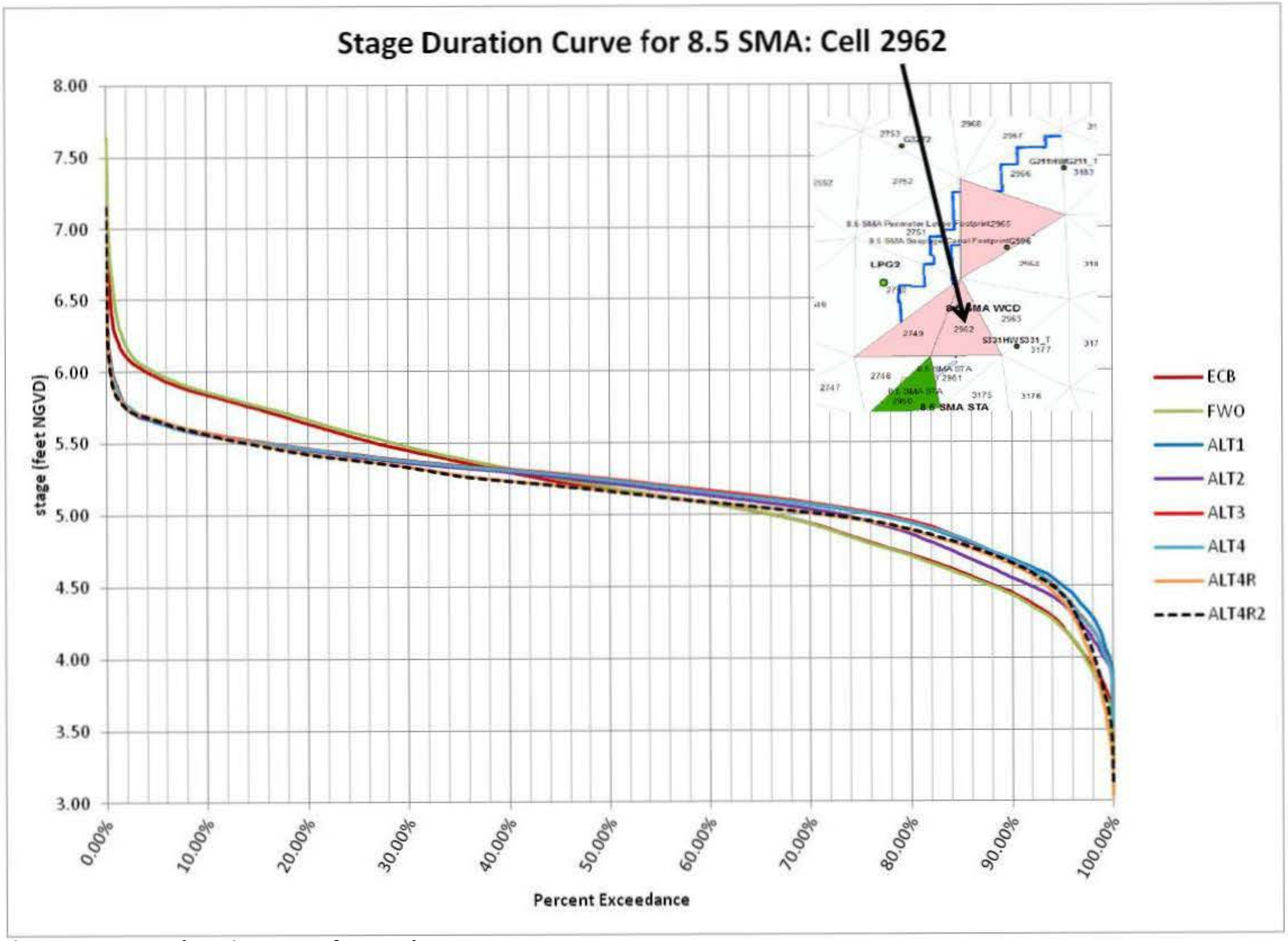


Figure 85: Stage duration curve for southeast 8.5 SMA

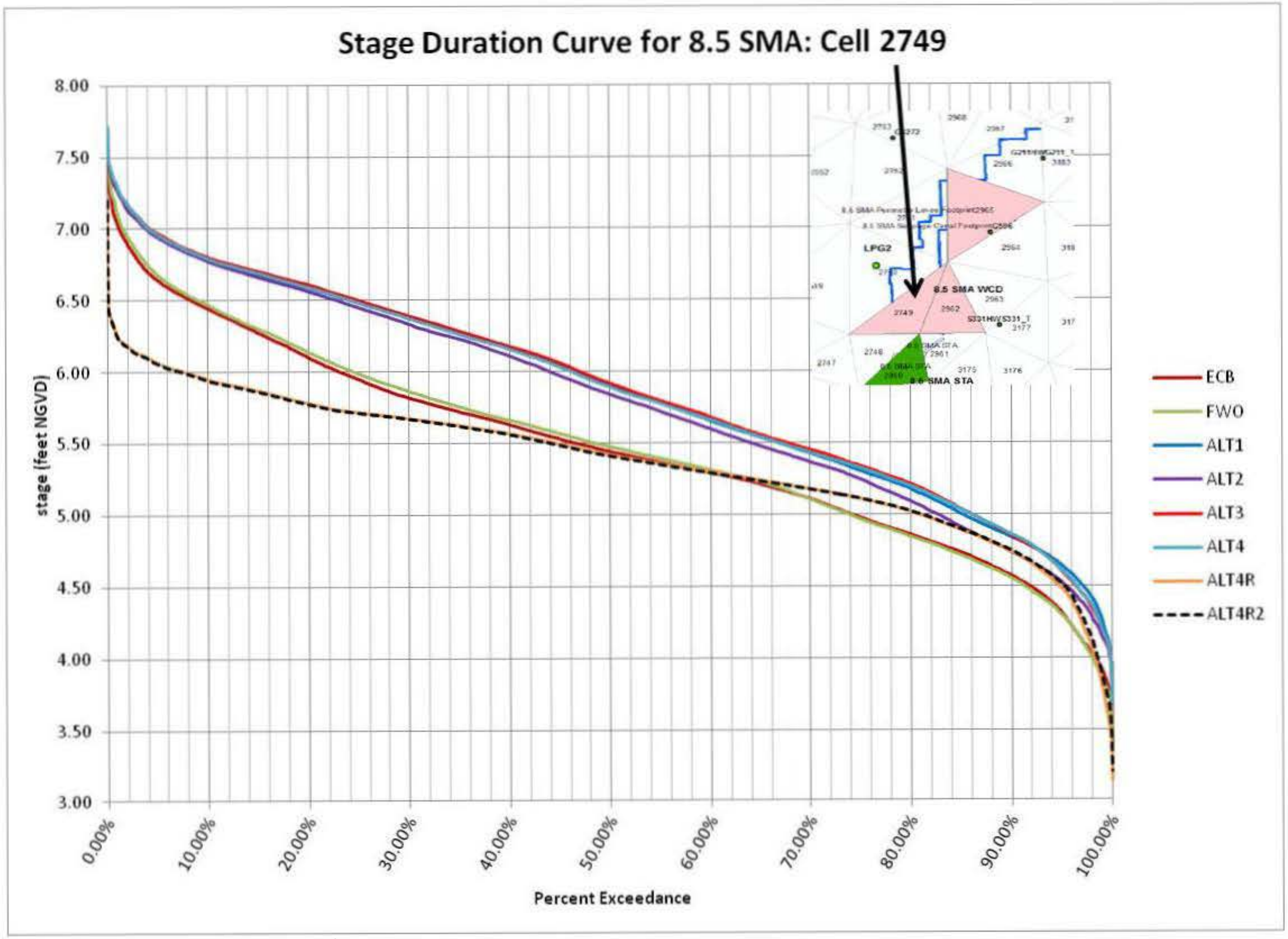


Figure 86: Stage duration curve for southwest 8.5 SMA

3.2.5. Additional RSM-GL Post-Processing for Structures and Detention Areas

RSM-GL daily output for structure discharges and water stages at monitoring gages are generated for the 1965-2005 period of simulation and tabulated using the USACE Hydrologic Engineering Center's Data Storage System (HEC-DSS). Due to the enormous volume of data included in the RSM-GL DSS files for the CEPP baselines and the CEPP alternatives, EN-W developed an additional suite of post-processed RSM-GL graphics to facilitate review of the preliminary Blue Line and Yellow Line screening modeling and the final array modeling by the CEPP water supply and flood control (WS/FC) technical sub-team. The primary assessment focus of the CEPP WS/FC sub-team was the South Dade Conveyance System (SDCS), including the effects of controlled/uncontrolled increased seepage from WCA-3B and eastern ENP with implementation of CEPP components and operations; the seepage flux dynamics along the Yellow Line are directly correlated to increased flood control risk (too much increased seepage and/or too little active seepage management) and reduced water availability for water supply (too little increased seepage and/or too much active seepage management).

Using the list of critical flow structures that was identified by EN-W for CEPP and included in the average annual critical flows reports (units are kAF), flow duration curve graphics were generated by EN-W for each of these critical structures to quantify the degree to which existing and/or proposed structure design capacities are sufficient for achievement of CEPP objectives, as well as the relative differences between the screening simulations and final alternatives. For reference, the critical flows reports generated for the final array of alternatives are provided in Table 7 (ECB, FWO, Alternatives 1 through 4) and Table 8 (ECB, FWO, Alternative 4R, Alternative 4R2). Stage duration curve graphics were also generated by EN-W for the 8.5 SMA Detention Area, C-111 North Detention Area, C-11 South Detention Area, and the Frog Pond Detention Area, to assess the relative differences in utilization of these storage areas for which standard model output graphics were not otherwise available. Several of the EN-W flow duration curves and stage duration curves were particularly utilized by the CEPP WS/FC sub-team during sub-team review of the final array modeling, and a selected sub-set of these graphics are provided in Figures 87 through 130. For each structure or detention area, two graphics are provided: the first graphic for each figure (part A) displays the ECB, FWO, and Alternatives 1 through 4R; the second graphic for each figure (part B) displays the ECB, 2012EC, IORBL1, Alternative 4R, and Alternative 4R2 (the 2012 Existing Condition Baseline [2012EC] and the Initial Operating Regime Baseline [IORBL1] simulations were specifically developed for the CEPP Savings Clause and Project Assurances assessments, and these simulations are discussed in more detail in Annex B of the CEPP PIR main report). The structures are generally sequenced from north to south and west to east, beginning with S-151. Aside from the unprocessed DSS output files, these flow duration curve and stage duration curve graphics are not otherwise available in the posted RSM-GL standard model output. On these graphics, absence of a legend entry for one or more base conditions or alternatives indicates that the particular structure was not included for the absent simulation. Graphic colors are automatically assigned by the post-processing script in sequential order, and graphics corresponding to structures or detention areas that are not included in all CEPP simulations will display with a different color scheme.

Table 7: RSM-GL critical flows report for CEPP baselines and Alternative 1 through 4

Comparison of Mean Annual Structure Discharges
for the period 1/1/1965 to 12/31/2005

	ECB	FWO	ALT1	ALT2	ALT3	ALT4
WCA-2A						

S10REG	282.44	268.20	265.89	265.91	265.92	265.89
S6FC	0.09	0.00	0.00	0.00	0.00	0.00
S6WS	15.30	12.89	14.56	14.56	14.56	14.56
S7FC	99.63	64.12	14.44	14.44	14.44	14.44
S7 WS	1.11	1.26	1.34	1.34	1.34	1.34
STA2O+BYP2N	221.06	373.51	217.65	217.65	217.65	217.65
STA2O+BYP2S	9.15	7.51	0.00	0.00	0.00	0.00
L6DIV	0.00	0.00	156.52	156.52	156.52	156.52
WCA-3B						

S151FC	238.53	231.53	87.19	68.36	85.61	89.11
S151WS	89.47	95.27	84.37	82.22	91.20	95.59
S31FC	30.68	30.09	27.86	44.82	25.97	24.51
S31WS	0.13	0.07	0.04	0.04	0.05	0.07
S337FC	9.98	9.00	0.00	0.00	0.00	0.00
S337WS	83.23	88.41	74.91	67.59	76.56	85.51
S355A	5.99	0.95	3.54	12.02	1.09	0.16
S355B	4.96	0.70	1.40	8.34	0.68	0.01
L29SA	-9.01	-9.01	-9.01	9.80	-9.01	-9.01
L29PA	-9.01	-9.01	-9.01	-9.01	242.19	-9.01
L29PB	-9.01	-9.01	-9.01	-9.01	169.18	-9.01
WCA-3A/L-29						

EASTERN_HRF	0.00	0.00	0.00	54.40	54.40	54.40
NWA3A_L28	161.44	200.49	871.66	697.85	697.85	697.85
S8FC	501.00	336.93	86.76	206.17	206.17	206.17
S8WS	28.46	30.76	0.00	0.00	0.00	0.00
S150	0.00	0.00	28.30	28.55	28.65	28.37
S140	190.51	191.37	279.33	214.82	214.81	214.86
S9	166.93	142.32	133.92	137.27	131.01	132.42
S11	382.13	460.13	287.80	287.43	287.35	287.42
S333	129.72	137.15	667.28	626.32	354.42	522.95
S334FC	44.11	45.42	0.00	0.00	0.00	0.00
S334WS	-9.01	-9.01	-9.01	-9.01	-9.01	-9.01
S335	89.63	87.95	92.83	158.96	99.47	99.56
S343	33.70	25.40	26.32	24.98	27.96	26.90
S344	19.23	15.86	16.44	15.60	17.46	16.80
S345A	-9.01	-9.01	-9.01	-9.01	-9.01	-9.01
S345B	-9.01	-9.01	-9.01	-9.01	-9.01	-9.01
S345C	-9.01	-9.01	-9.01	-9.01	52.52	-9.01

S345D	-9.01	-9.01	-9.01	129.77	106.26	82.11
S345E	-9.01	-9.01	126.85	59.60	51.12	-9.01
S345G	-9.01	-9.01	-9.01	-9.01	-9.01	96.93
S345F	-9.01	-9.01	-9.01	75.64	79.39	82.39
S356	0.00	0.00	63.64	83.92	29.88	28.58
S12A	37.33	29.76	15.25	14.54	16.76	15.87
S12B	98.56	92.16	46.57	44.90	51.06	48.26
S12C	172.91	242.85	150.81	143.93	157.56	152.74
S12D	384.87	320.39	204.73	194.48	211.93	206.29
	693.67	685.16	417.36	397.85	437.31	423.16
ENP & Detention Areas						
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L31PA	-9.01	-9.01	98.42	-9.01	-9.01	-9.01
L31PB	-9.01	-9.01	159.20	17.79	-9.01	-9.01
S332B	59.37	65.57	61.69	47.19	48.38	61.20
S332BN	42.66	51.77	48.16	37.36	38.37	47.91
S332C	75.12	87.77	74.12	55.45	57.18	74.39
S332D	97.57	107.09	101.76	106.62	107.83	100.35
South Dade						
--						
G211FC	110.17	110.78	2.90	0.66	0.08	6.83
G211WS	60.48	62.41	57.51	60.24	58.58	58.09
S336	5.93	5.71	4.04	4.21	5.73	6.35
S338	58.89	57.13	51.17	53.19	29.02	32.97
S357	2.66	3.27	47.20	45.18	48.22	47.31
S331FC	164.10	164.94	87.39	80.43	80.38	86.29
S331WS	61.41	62.58	53.02	57.35	54.70	54.35
S194	20.98	25.82	15.89	15.63	16.86	18.53
S196	8.99	13.72	6.59	6.09	6.08	7.14
S176	50.25	42.98	42.28	43.04	43.06	42.03
S177	124.33	69.78	73.25	71.96	72.80	71.31
S18C	182.80	144.98	144.38	143.52	145.08	144.56
S197	16.49	6.66	7.91	7.85	8.12	8.25
CERP Impoundments						
--						
S504	-9.01	19.44	18.78	18.89	18.56	18.76
S509_P	-9.01	0.14	0.06	0.07	0.15	0.05
S510	-9.01	0.04	0.04	0.04	0.04	0.04
S525A_P	-9.01	5.39	5.46	5.47	5.48	5.46
S526A_C	-9.01	12.90	12.93	12.94	12.94	12.93
S199	-9.01	32.50	30.60	33.00	33.60	28.20
S200	-9.01	89.90	91.80	89.29	89.53	90.02

Run date: December 19, 2012
NOTE: Values of -9.01 indicate structure not found!!

Table 8: RSM-GL critical flows report for CEPP baselines, Alternative 4R, and Alternative 4R2

Comparison of Mean Annual Structure Discharges (kacft/yr)
for the period 1/1/1965 to 12/31/2005

	ECB	FWO	ALT4R	ALT4R2
WCA-2A				

S10REG	282.4	268.2	265.7	266.0
S6FC	0.1	0.0	0.0	0.0
S6WS	1.1	1.3	1.3	1.3
S7FC	99.6	64.1	14.5	55.6
S7 WS	15.3	12.9	14.6	12.3
STA2O+BYP2N	221.1	373.5	229.6	235.8
STA2O+BYP2S	9.2	7.5	0.0	0.0
L6DIV	0.0	0.0	144.7	147.9
WCA-3B				

S151FC	238.5	231.5	89.9	89.8
S151WS	89.5	95.3	82.1	76.7
S31FC	30.7	30.1	32.2	33.4
S31WS	0.1	0.1	0.1	0.1
S337FC	10.0	9.0	0.0	0.0
S337WS	83.2	88.4	74.4	70.1
S355A	6.0	1.0	1.9	2.1
S355B	5.0	0.7	0.8	0.9
L29SA	-9.01	-9.01	-9.01	-9.01
L29PA	-9.01	-9.01	-9.01	-9.01
L29PB	-9.01	-9.01	-9.01	-9.01
L29_DIVIDE	-9.01	-9.01	38.0	37.8
WCA-3A/L-29				

EASTERN_HRF	0.0	0.0	0.0	0.0
NWA3A_L28	161.4	200.5	757.0	757.0
S8FC	501.0	336.9	161.3	120.1
S8WS	28.5	30.8	0.0	0.0
S150	0.0	0.0	57.4	57.7
S140	190.5	191.4	197.3	199.0
S9	166.9	142.3	134.7	145.3
S11	382.1	460.1	289.8	323.1
S333	129.7	137.2	456.1	457.0
S334FC	44.1	45.4	0.0	0.0
S334WS	-9.01	-9.01	-9.01	-9.01
S335	89.6	88.0	82.5	85.3
S343	33.7	25.4	27.3	27.4
S344	19.2	15.9	17.1	17.1
S345A	-9.01	-9.01	-9.01	-9.01
S345B	-9.01	-9.01	-9.01	-9.01

S345C	-9.01	-9.01	-9.01	-9.01
S345D	-9.01	-9.01	132.6	132.9
S345E	-9.01	-9.01	-9.01	-9.01
S345G	-9.01	-9.01	112.9	113.5
S345F	-9.01	-9.01	130.7	131.4
S356	0.0	0.0	67.4	63.9
S12A	37.3	29.8	13.9	14.0
S12B	98.6	92.2	43.3	43.7
S12C	172.9	242.9	135.9	138.1
S12D	384.9	320.4	182.4	185.3
	693.7	685.2	375.5	381.1
ENP & Detention Areas				

L31PA	-9.01	-9.01	-9.01	-9.01
L31PB	-9.01	-9.01	-9.01	-9.01
S332B	59.4	65.6	65.7	63.1
S332BN	42.7	51.8	66.4	63.6
S332C	75.1	87.8	114.2	108.0
S332D	97.6	107.1	127.5	124.4
South Dade				

G211FC	110.2	110.8	64.0	57.2
G211WS	60.5	62.4	50.5	52.5
S336	5.9	5.7	4.6	4.3
S338	58.9	57.1	58.0	72.9
S357	2.7	3.3	59.5	58.9
S331FC	164.1	164.9	134.0	130.3
S331WS	61.4	62.6	49.8	51.4
S194	21.0	25.8	20.8	26.8
S196	9.0	13.7	8.8	13.6
S176	50.3	43.0	35.4	35.1
S177	124.3	69.8	80.3	79.6
S18C	182.8	145.0	157.8	155.6
S197	16.5	6.7	8.4	8.2
CERP Impoundments				

S504	-9.01	19.4	18.5	17.9
S509_P	-9.01	0.1	0.2	0.2
S510	-9.01	0.0	0.0	0.1
S525A_P	-9.01	5.4	5.5	5.5
S526A_C	-9.01	12.9	12.9	12.9
S199	-9.01	32.5	33.9	32.2
S200	-9.01	89.9	102.6	102.6

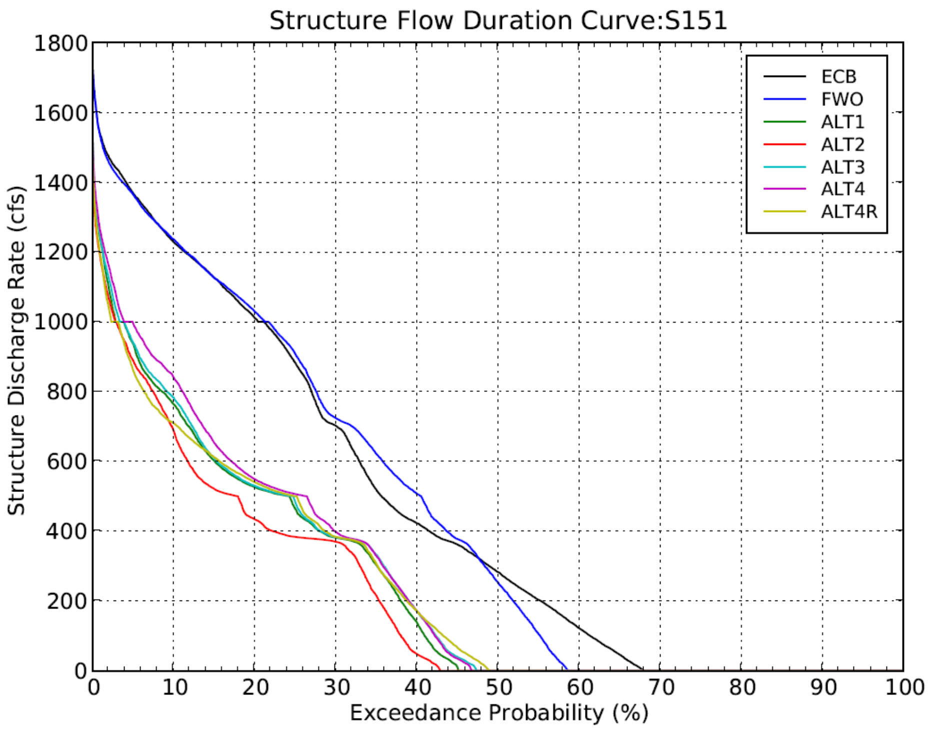


Figure 87A: Flow duration curve for WCA-3A outflow structure S-151

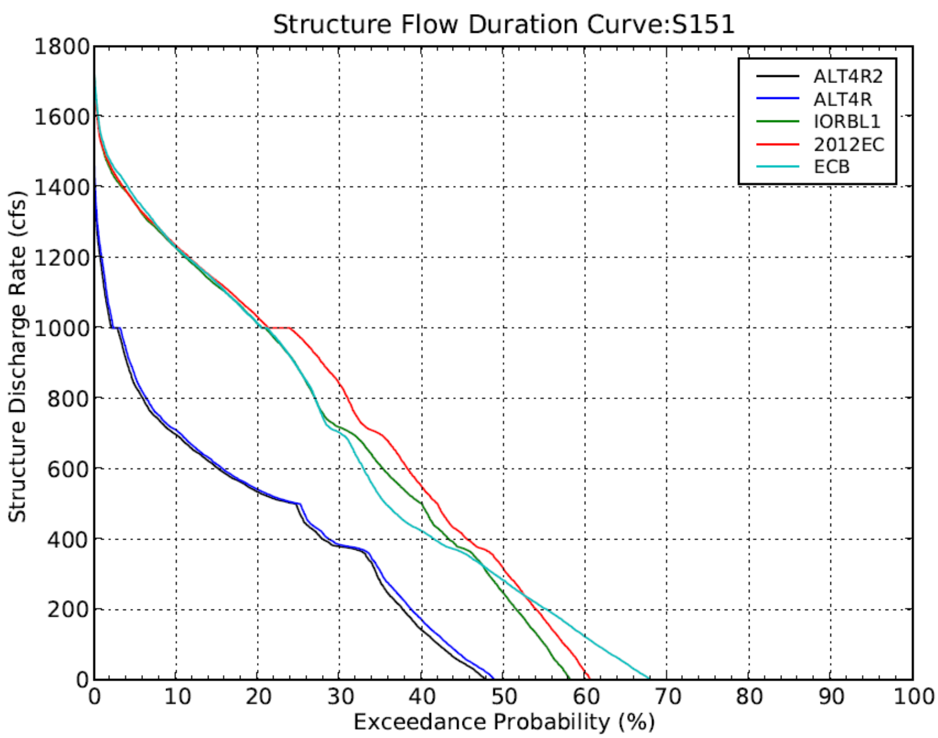


Figure 87B: Flow duration curve for WCA-3A outflow structure S-151

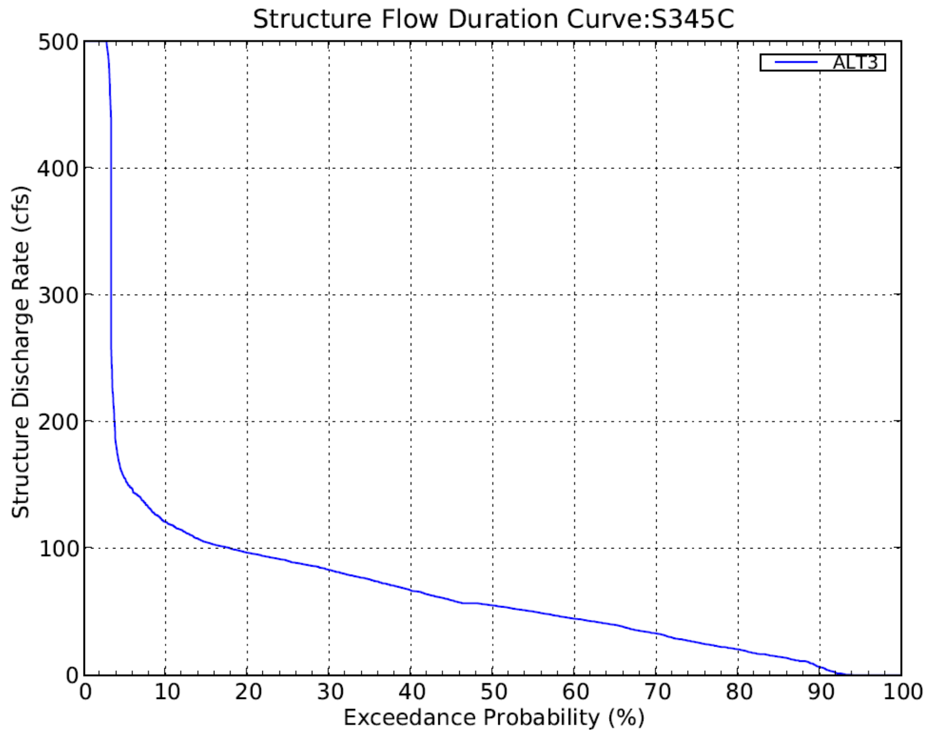


Figure 88A: Flow duration curve for WCA-3A outflow structure S-345C

Note: Structure S345C is not included in the simulations displayed in the part B graphics.

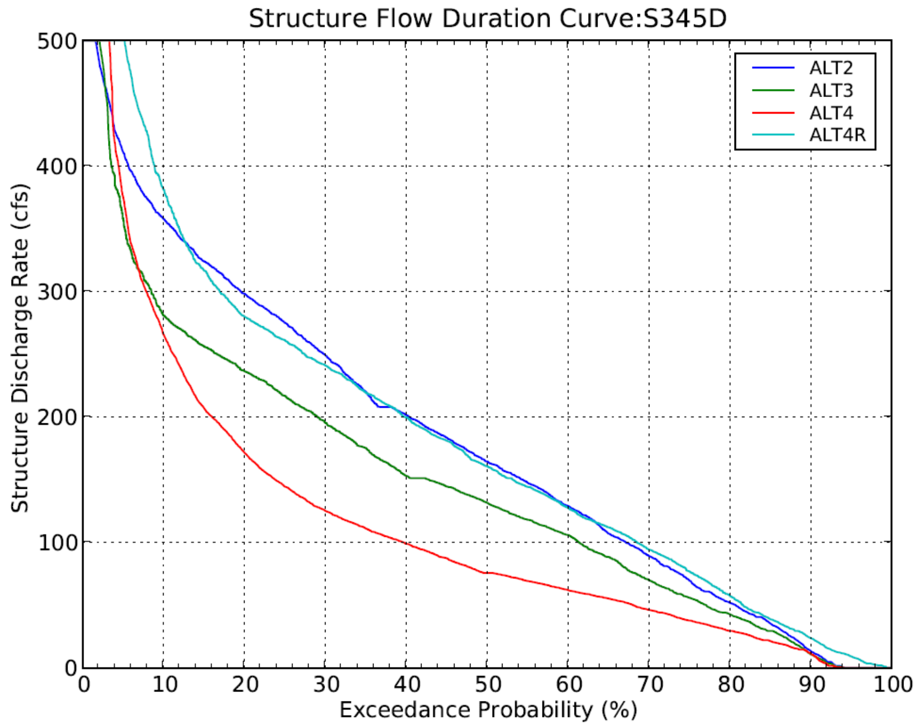


Figure 89A: Flow duration curve for WCA-3A outflow structure S-345D

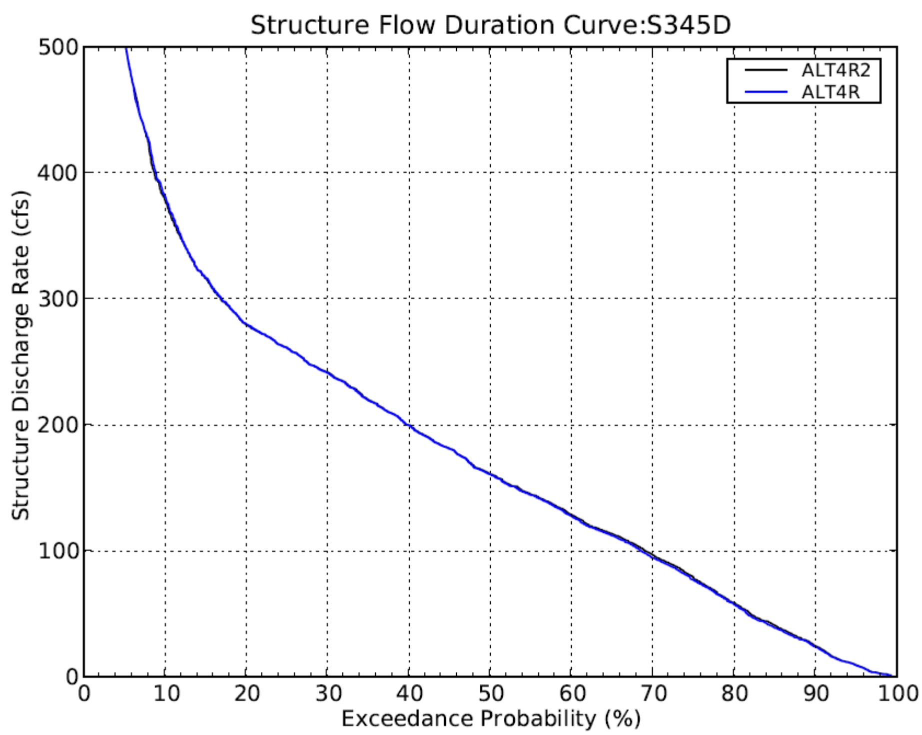


Figure 89B: Flow duration curve for WCA-3A outflow structure S-345D

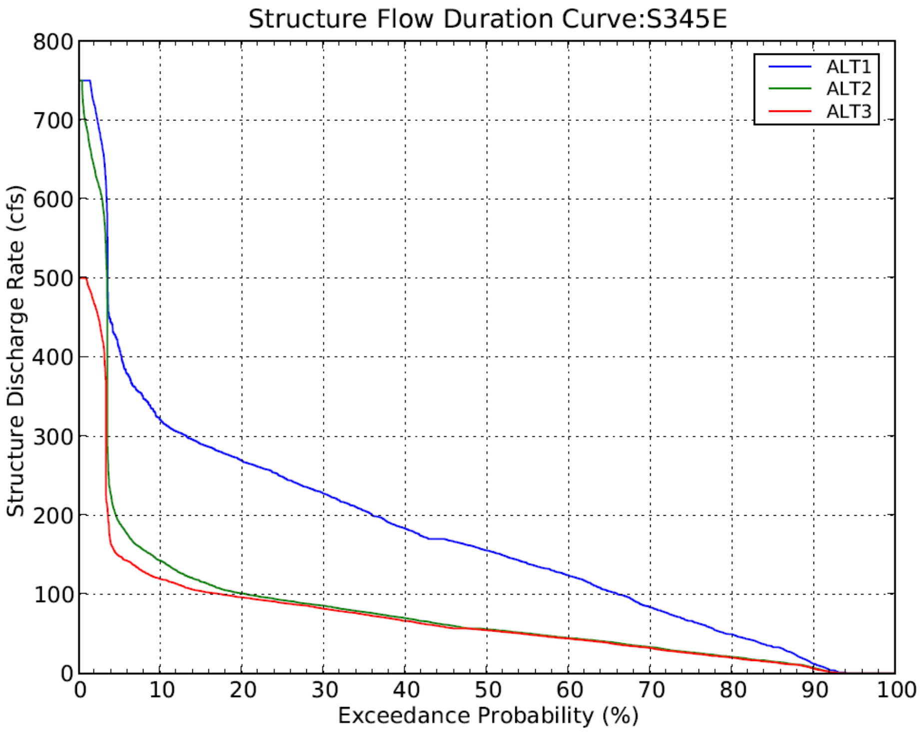


Figure 90A: Flow duration curve for WCA-3A outflow structure S-345E

Note: Structure S345E is not included in the simulations displayed in the part B graphics.

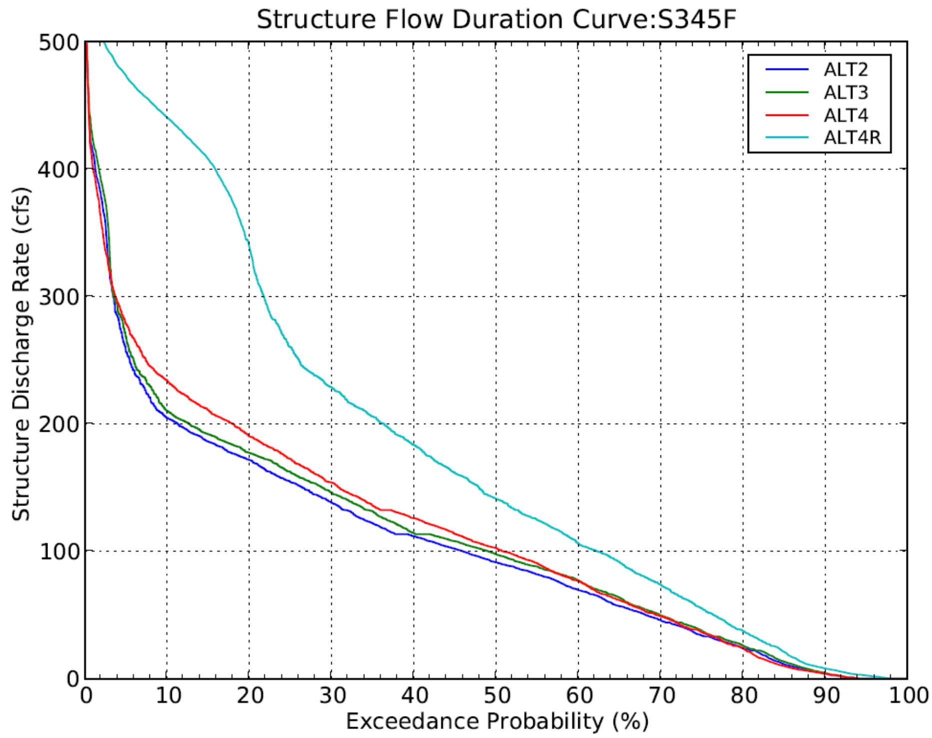


Figure 91A: Flow duration curve for WCA-3A outflow structure S-345F

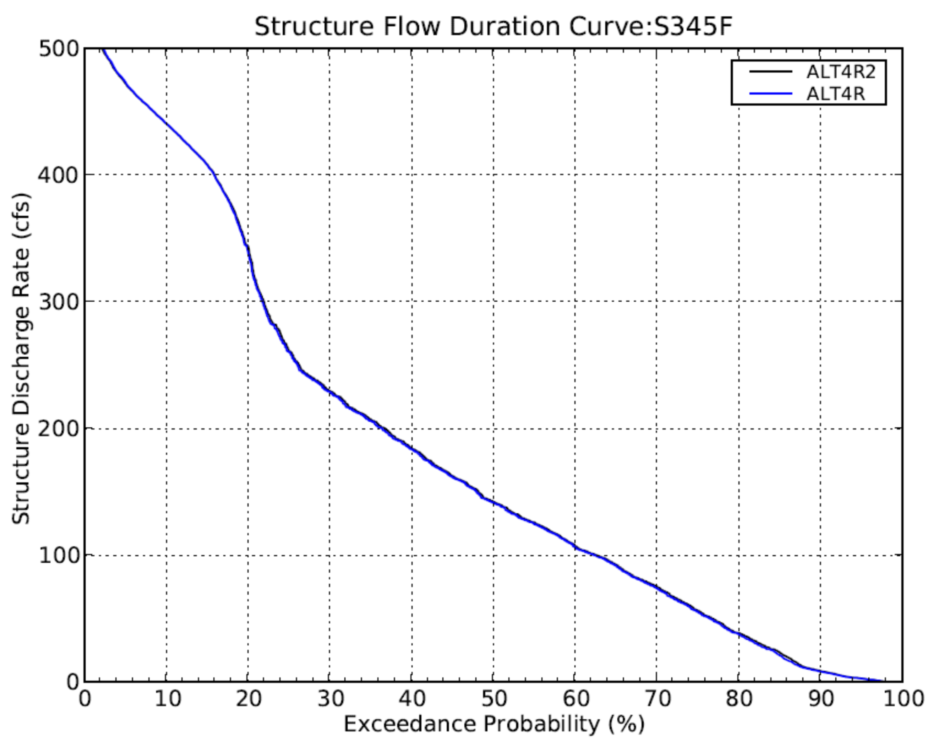


Figure 91B: Flow duration curve for WCA-3A outflow structure S-345F

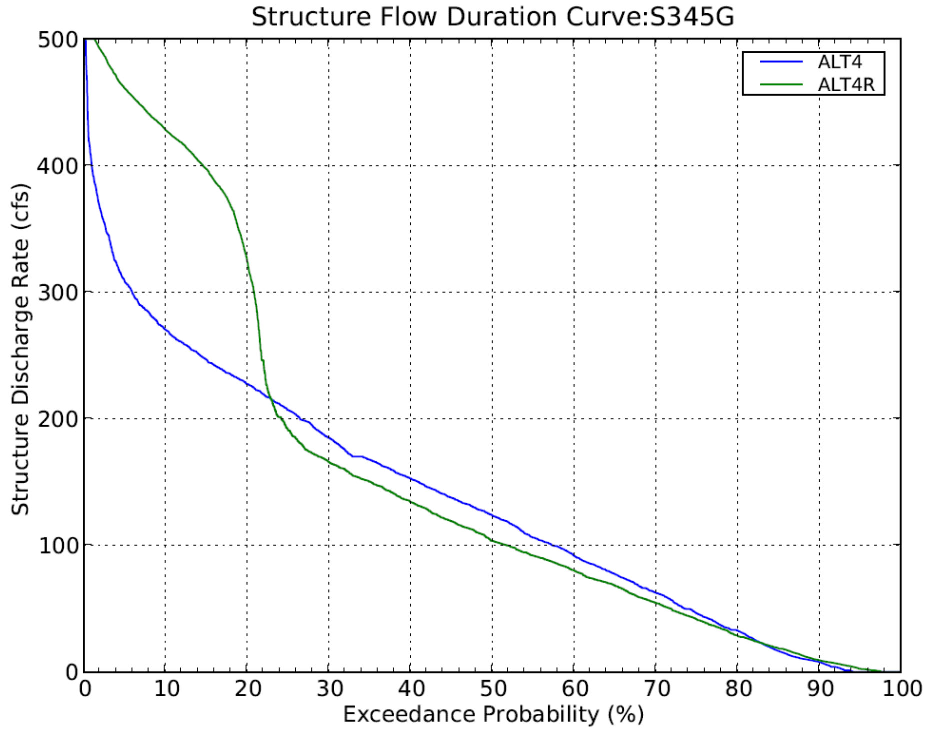


Figure 92A: Flow duration curve for WCA-3A outflow structure S-345G

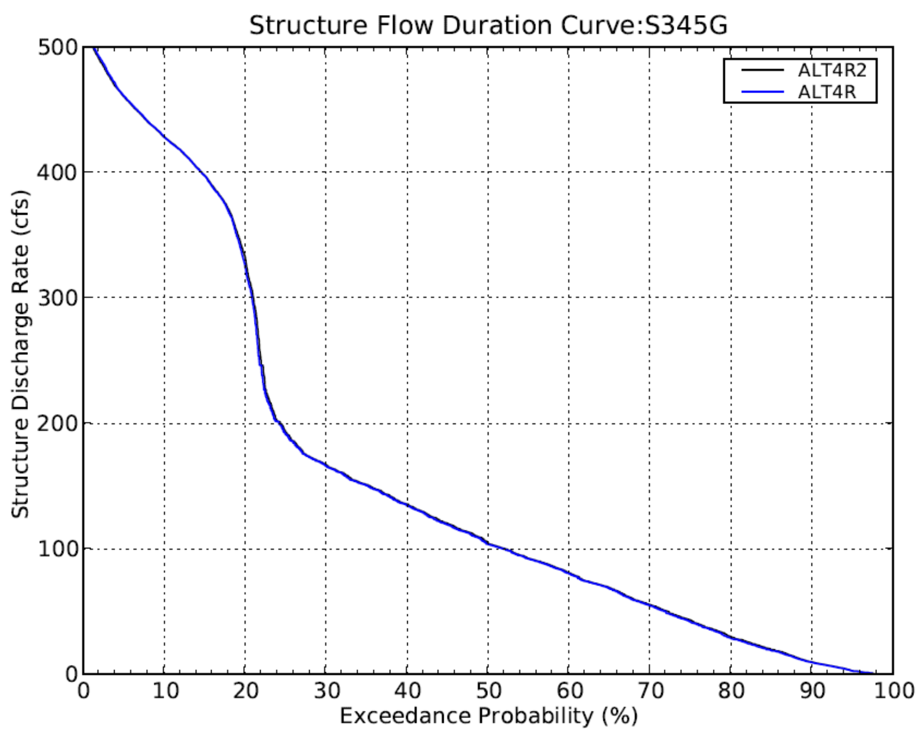


Figure 92B: Flow duration curve for WCA-3A outflow structure S-345G

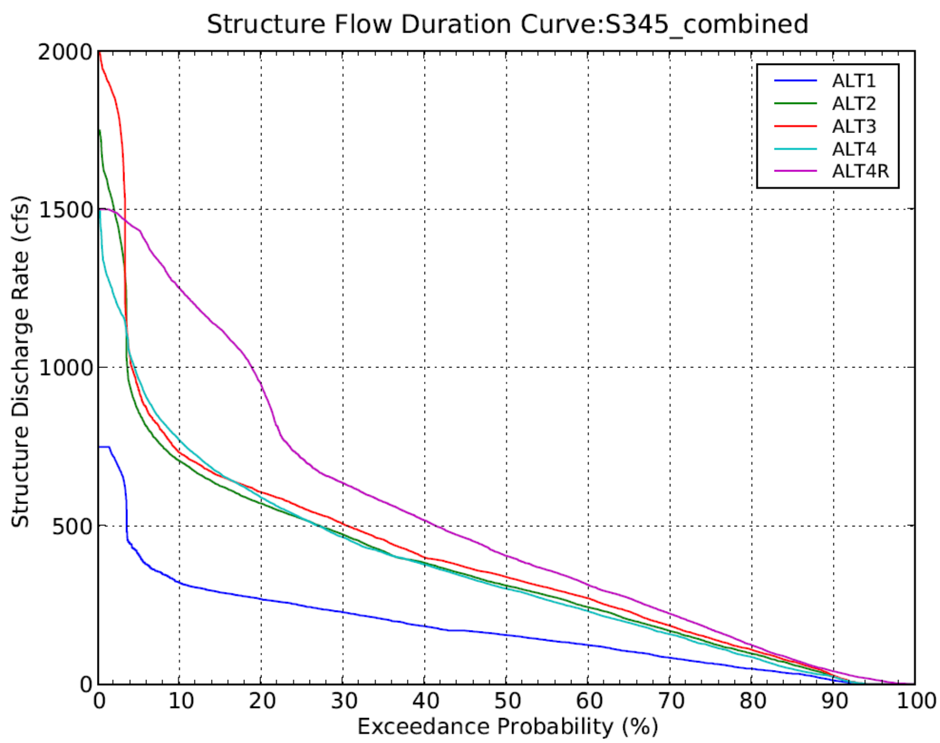


Figure 93A: Combined flow duration curve for WCA-3A outflow structures: S-345C, S-345D, S-345E, S-345F, and S-345G (CEPP proposed new WCA-3B inflow structures)

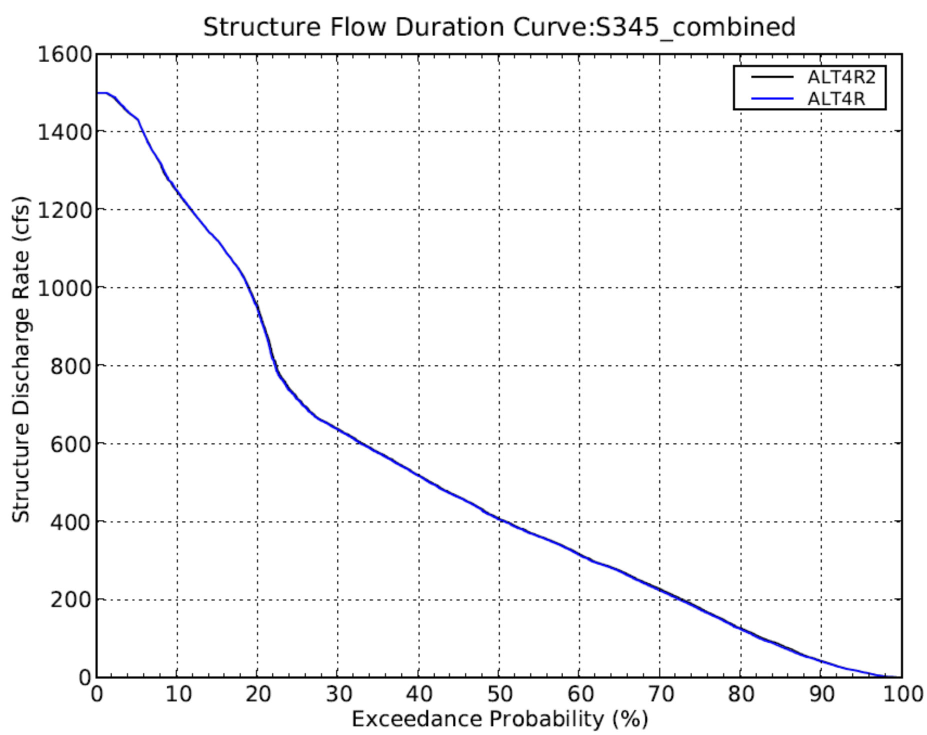


Figure 93B: Combined flow duration curve for WCA-3A outflow structures: S-345C, S-345D, S-345E, S-345F, and S-345G (CEPP proposed new WCA-3B inflow structures)

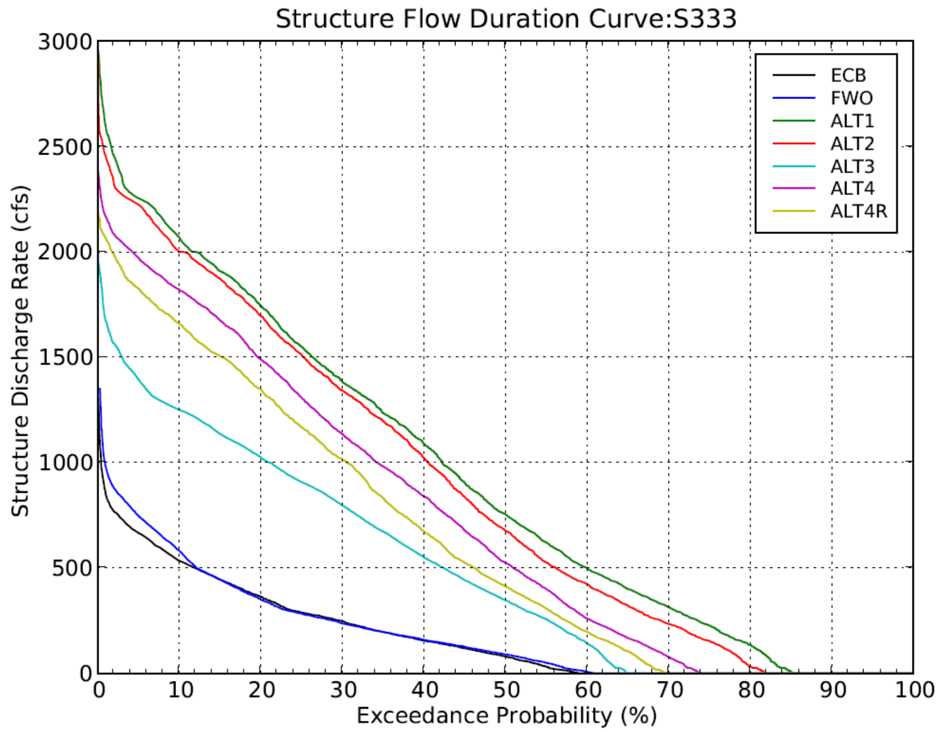


Figure 94A: Flow duration curve for WCA-3A outflow structure S-333

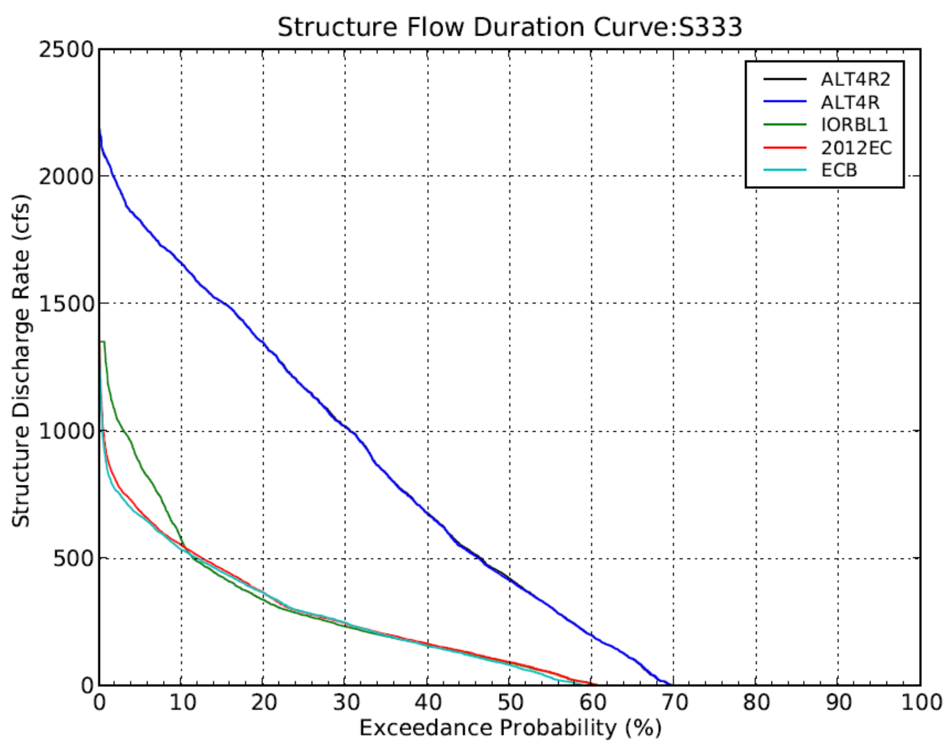


Figure 94B: Flow duration curve for WCA-3A outflow structure S-333

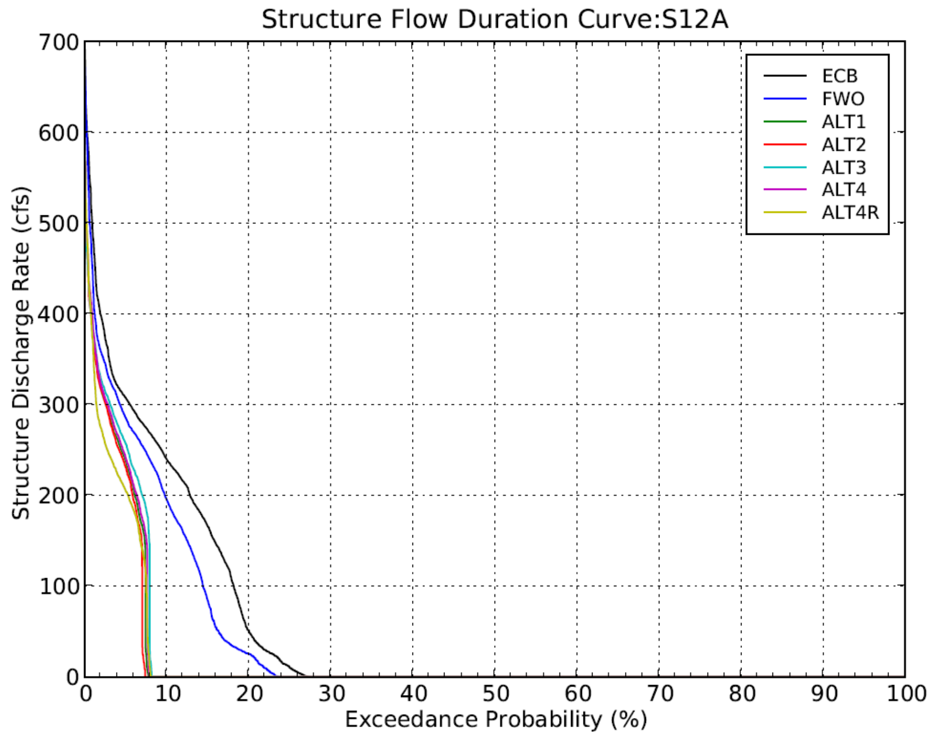


Figure 95A: Flow duration curve for WCA-3A outflow structure S-12A

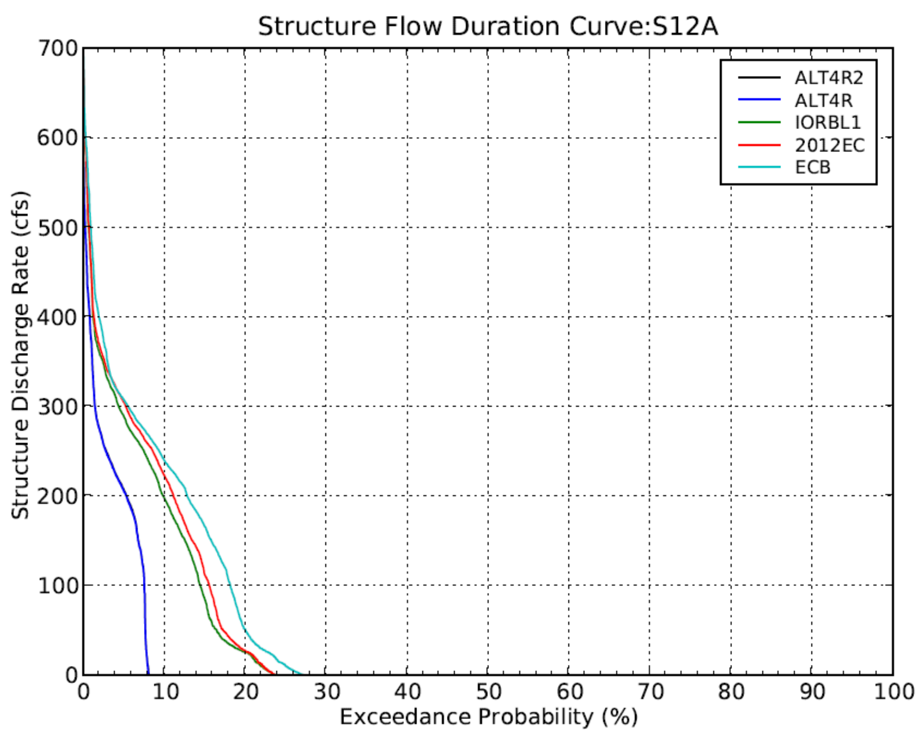


Figure 95B: Flow duration curve for WCA-3A outflow structure S-12A

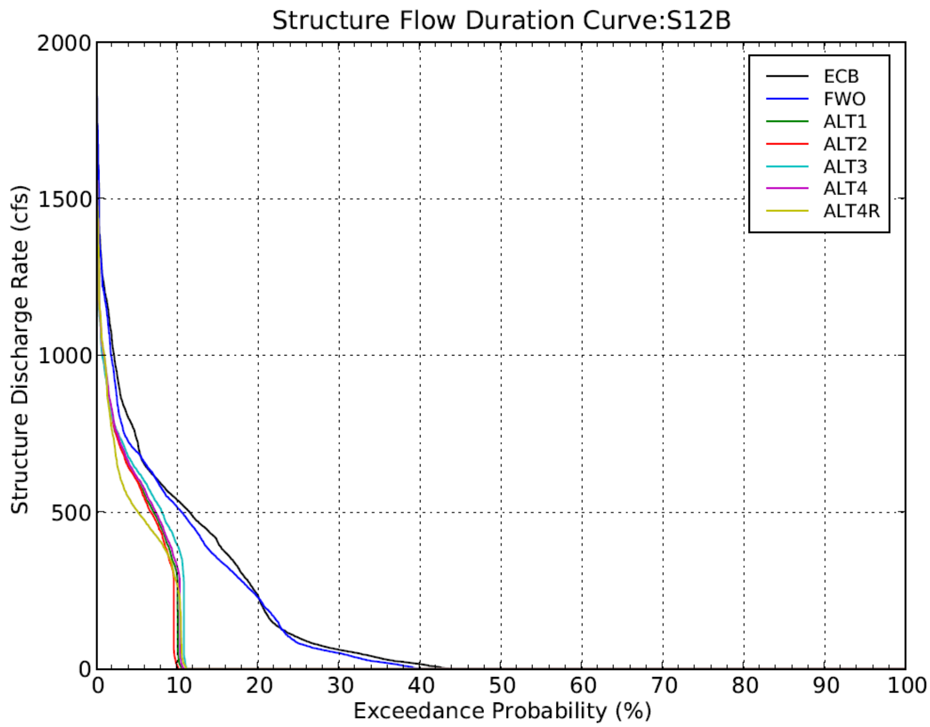


Figure 96A: Flow duration curve for WCA-3A outflow structure S-12B

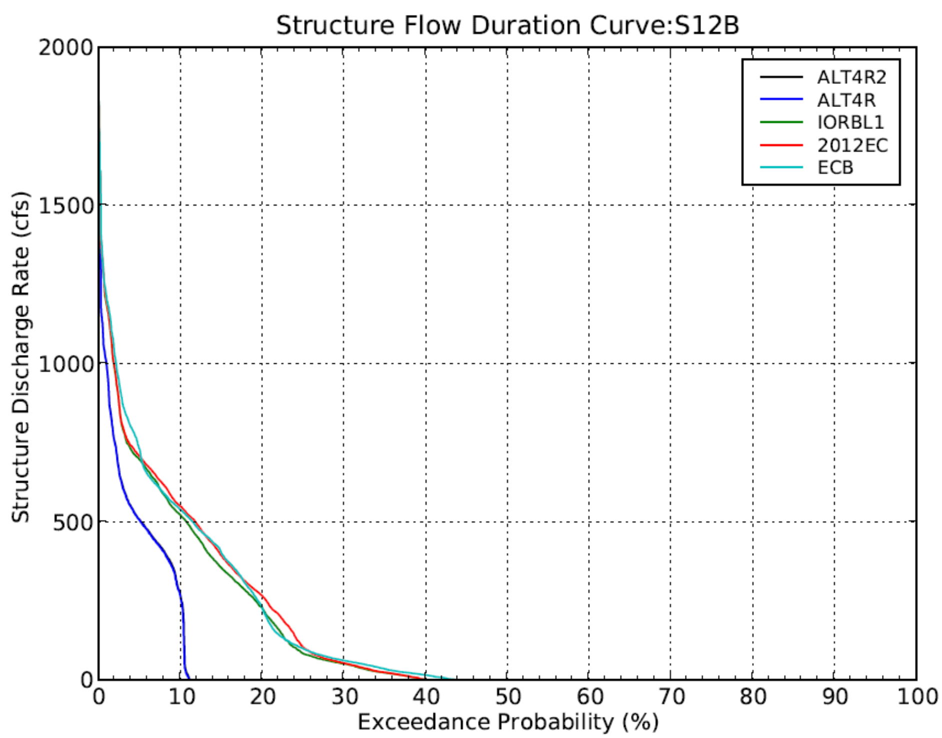


Figure 96B: Flow duration curve for WCA-3A outflow structure S-12B

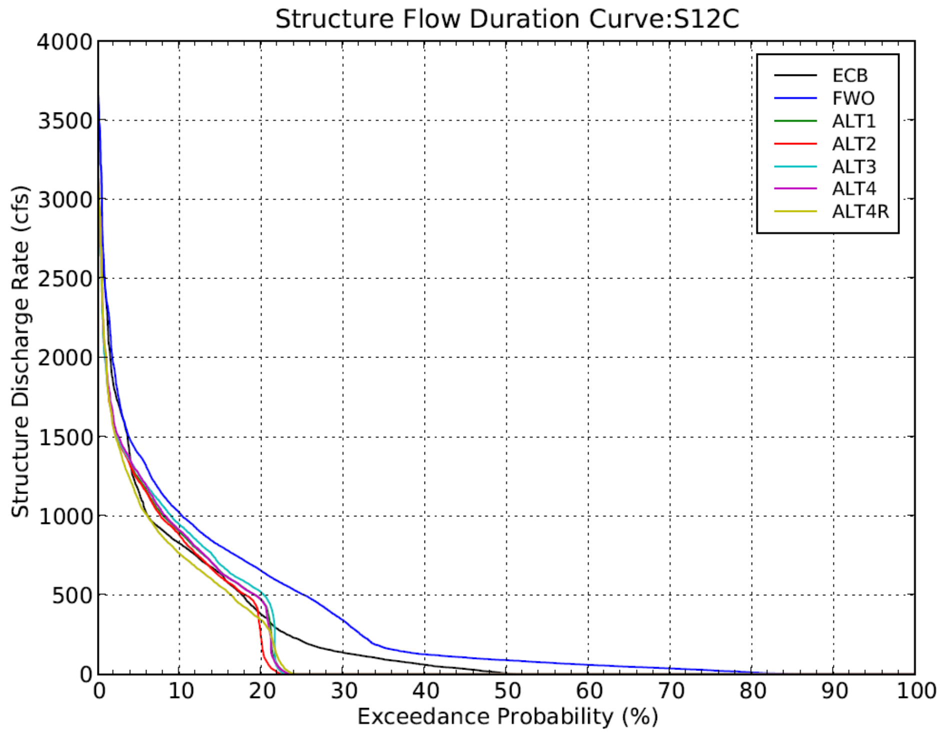


Figure 97A: Flow duration curve for WCA-3A outflow structure S-12C

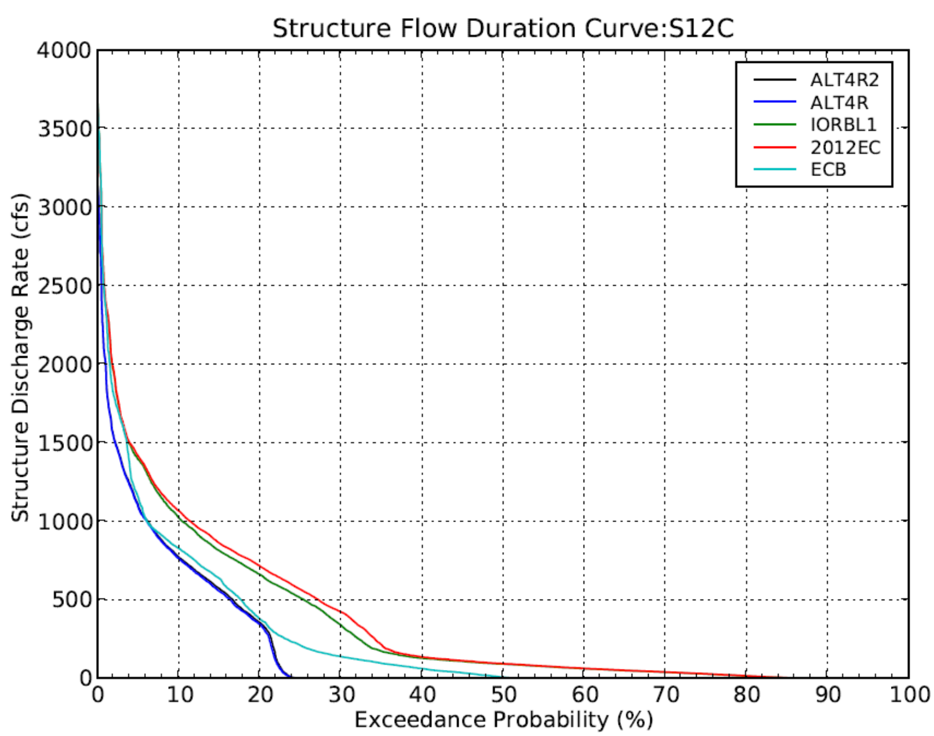


Figure 97B: Flow duration curve for WCA-3A outflow structure S-12C

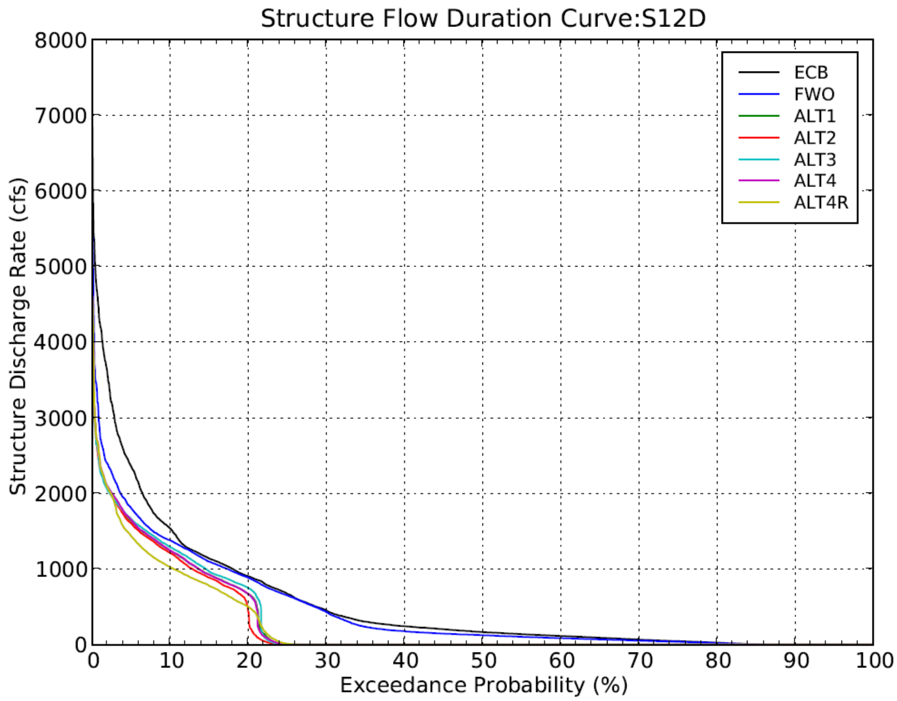


Figure 98A: Flow duration curve for WCA-3A outflow structure S-12D

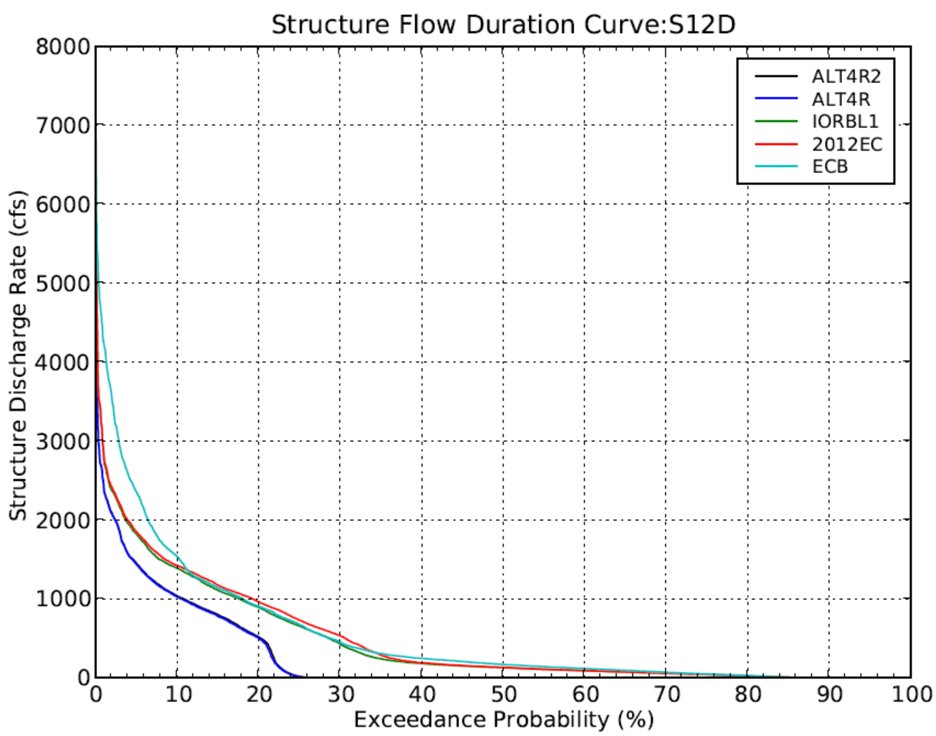


Figure 98B: Flow duration curve for WCA-3A outflow structure S-12D

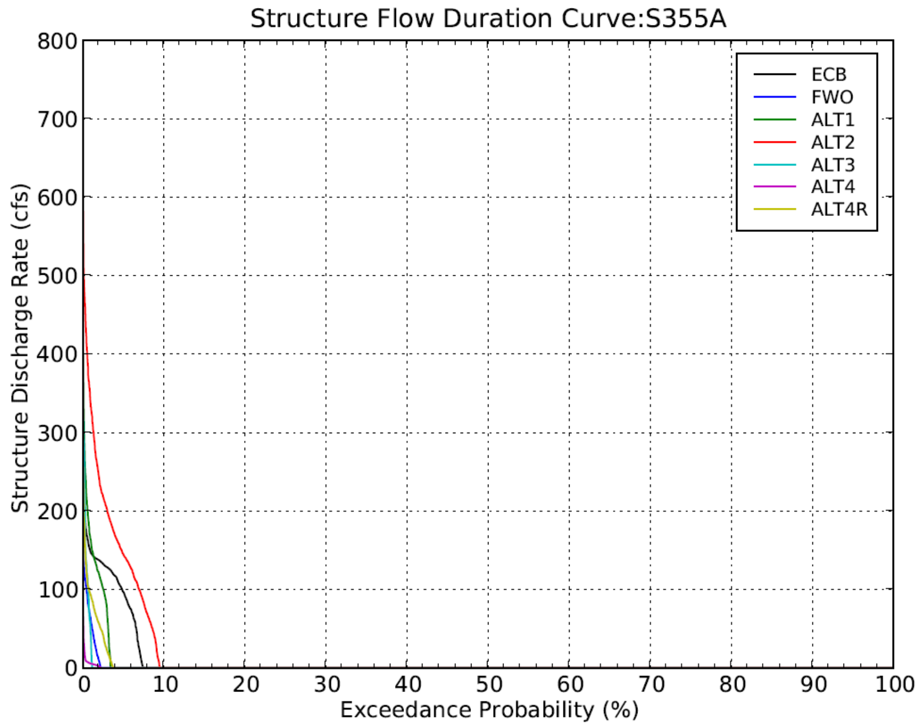


Figure 99A: Flow duration curve for WCA-3B outflow structure S-355A

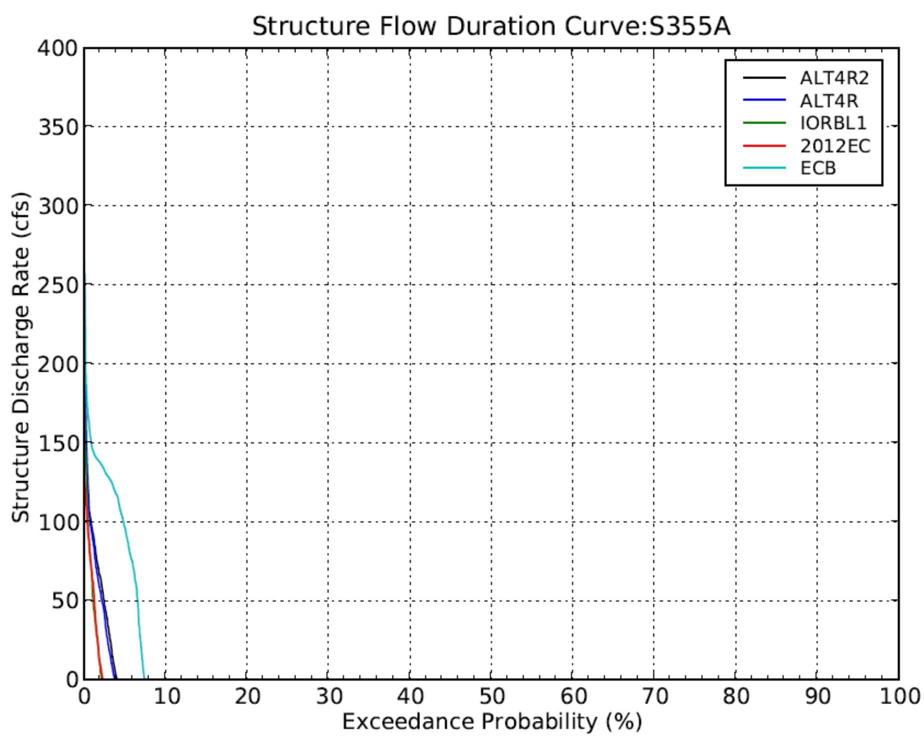


Figure 99B: Flow duration curve for WCA-3B outflow structure S-355A

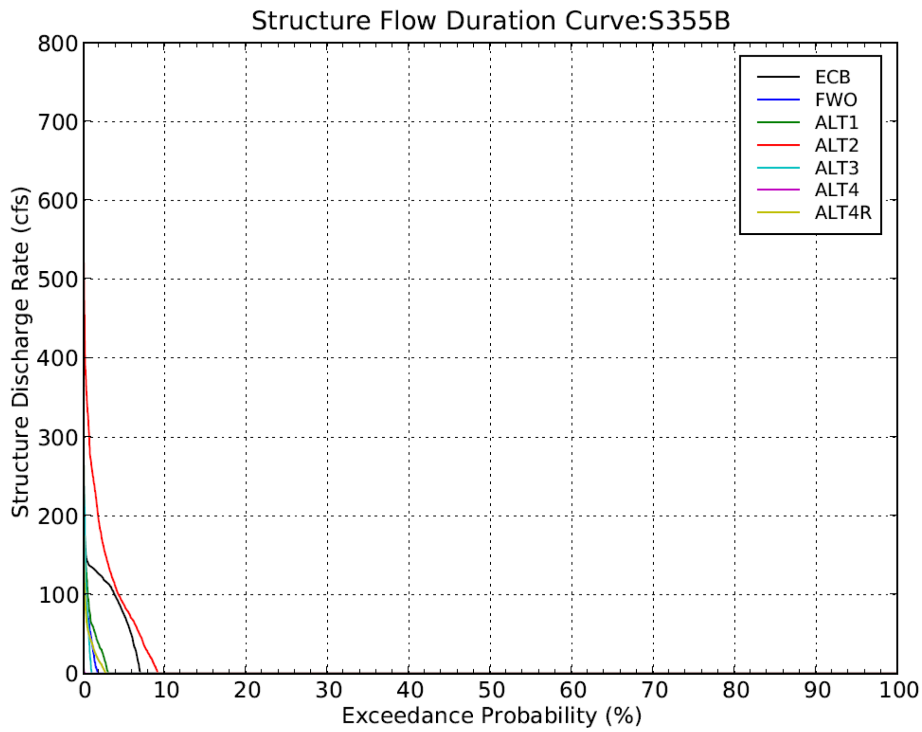


Figure 100A: Flow duration curve for WCA-3B outflow structure S-355B

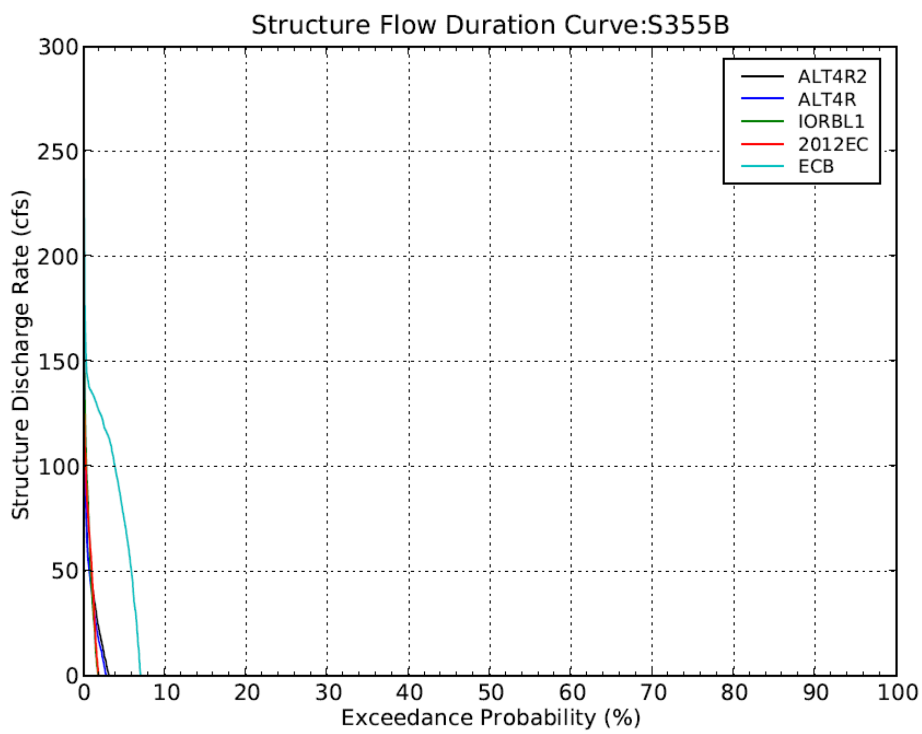


Figure 100B: Flow duration curve for WCA-3B outflow structure S-355B

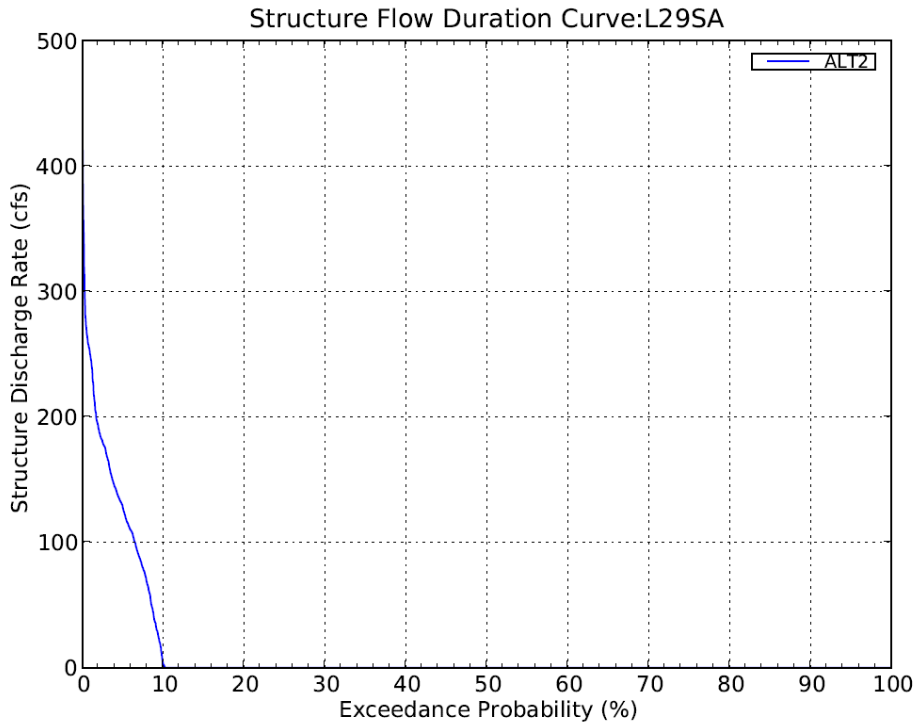


Figure 101A: Flow duration curve for WCA-3B outflow structure L-29SA

Note: Structure L-29SA is not included in the simulations displayed in the part B graphics.

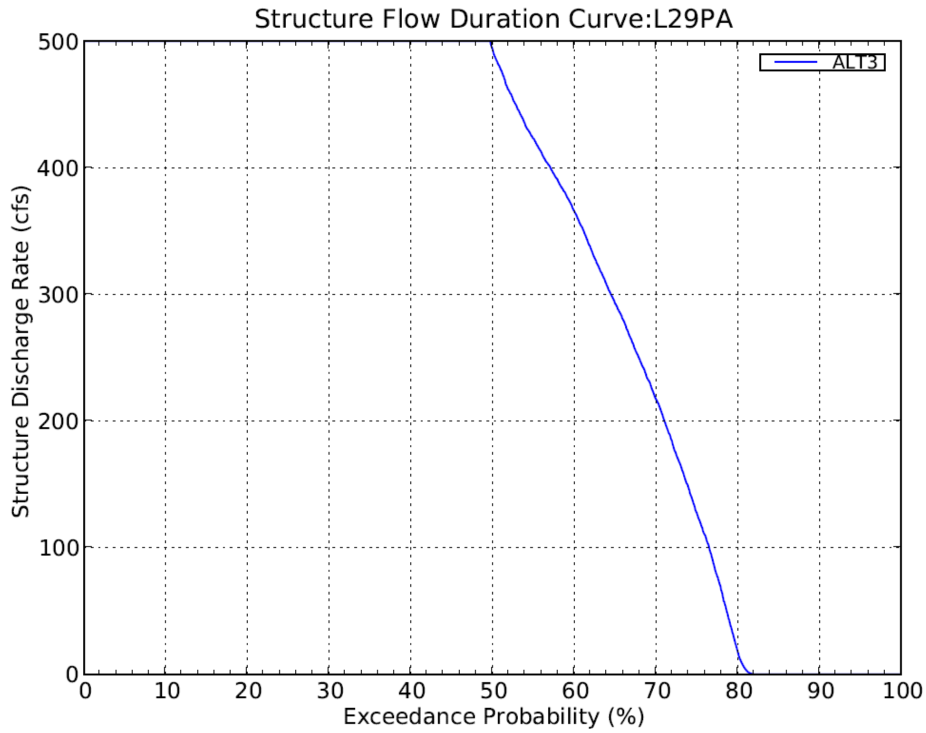


Figure 102A: Flow duration curve for WCA-3B outflow structure L-29PA

Note: Structure L-29PA is not included in the simulations displayed in the part B graphics.

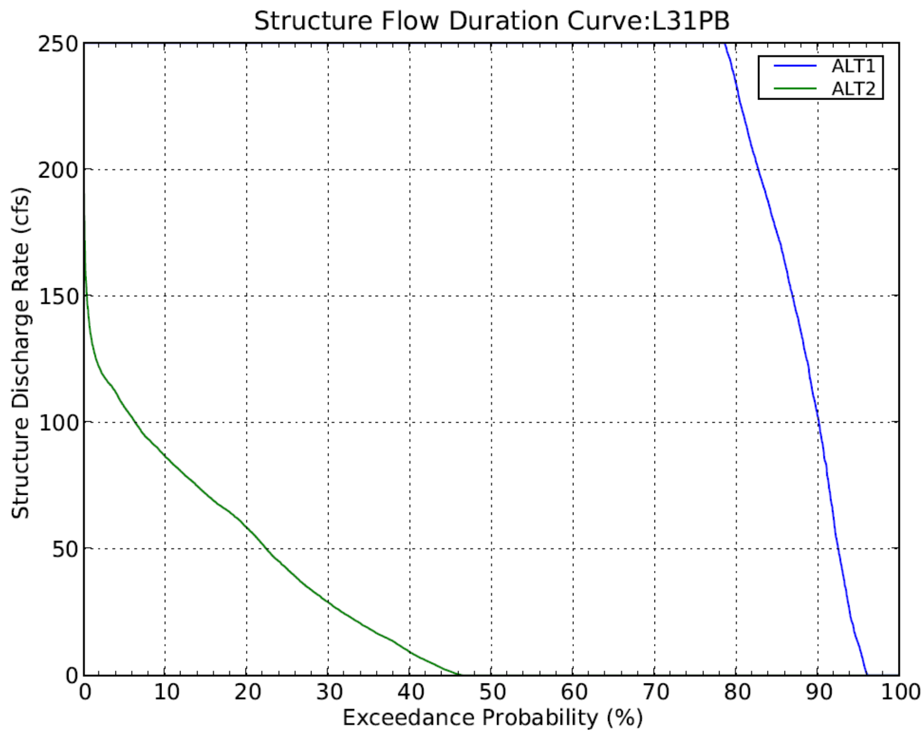


Figure 103A: Flow duration curve for WCA-3B outflow structure L-29PB

Note: Structure L-29PB is not included in the simulations displayed in the part B graphics.

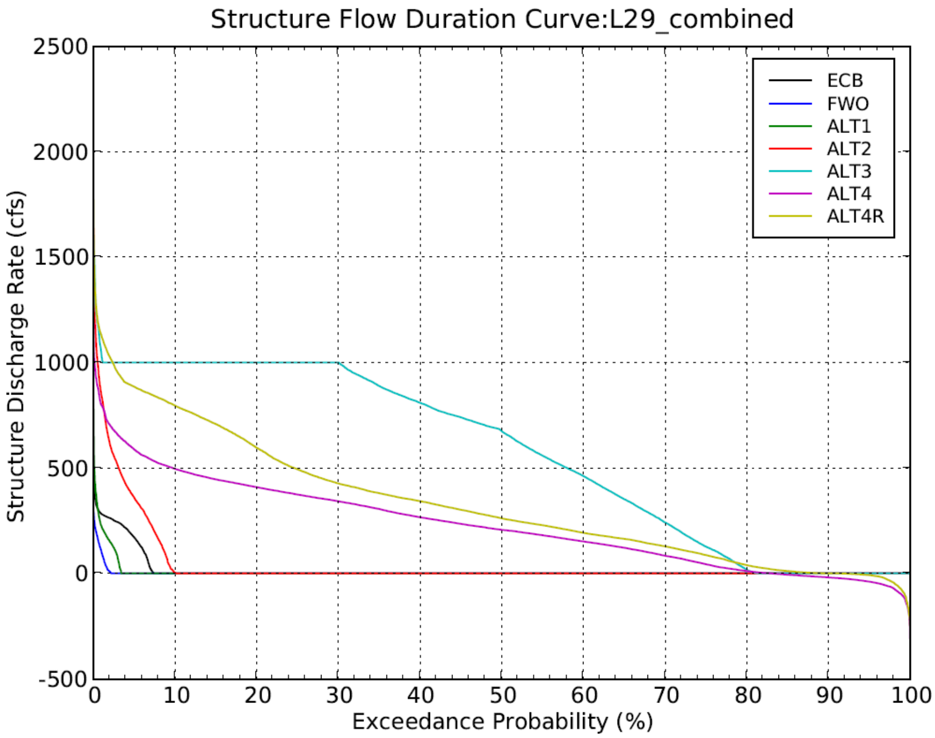


Figure 104A: Combined flow duration curve for WCA-3B outlet structures to L-29 Canal

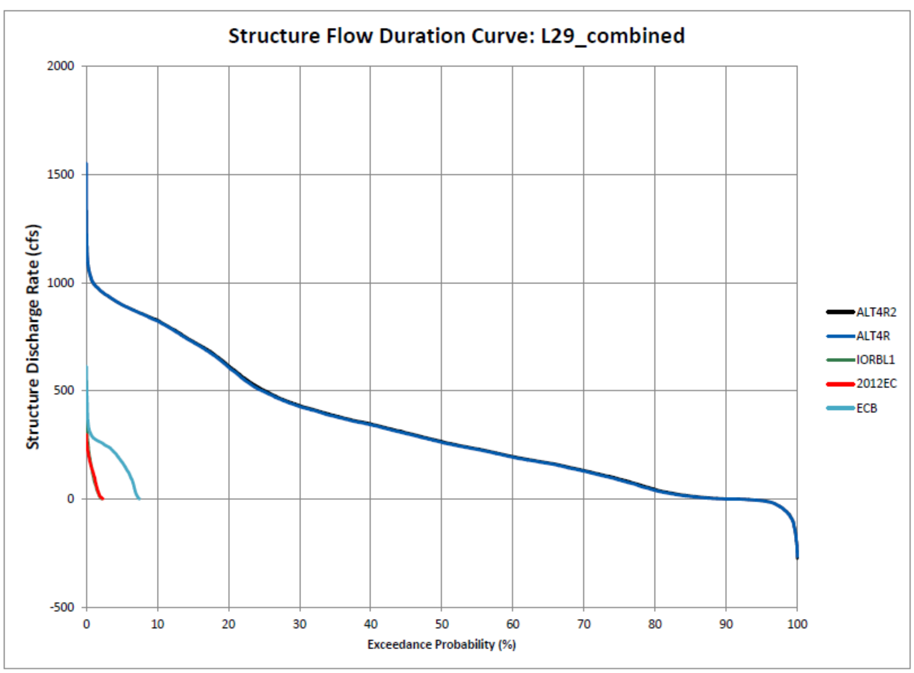


Figure 104B: Combined flow duration curve for WCA-3B outlet structures to L-29 Canal

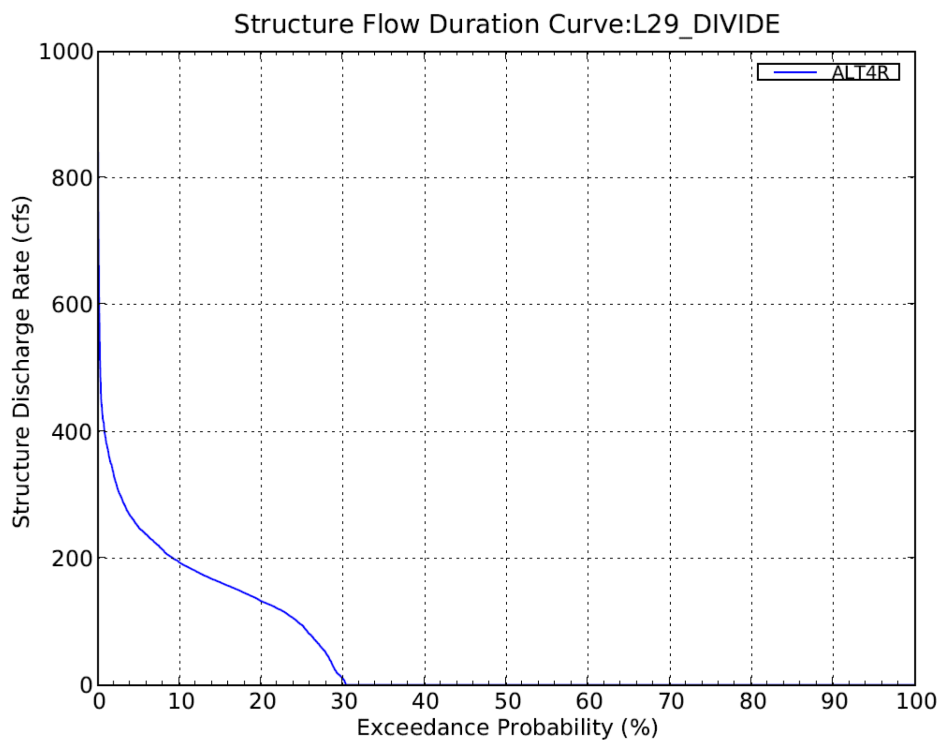


Figure 105A: Flow duration curve for L-29 Canal divide structure

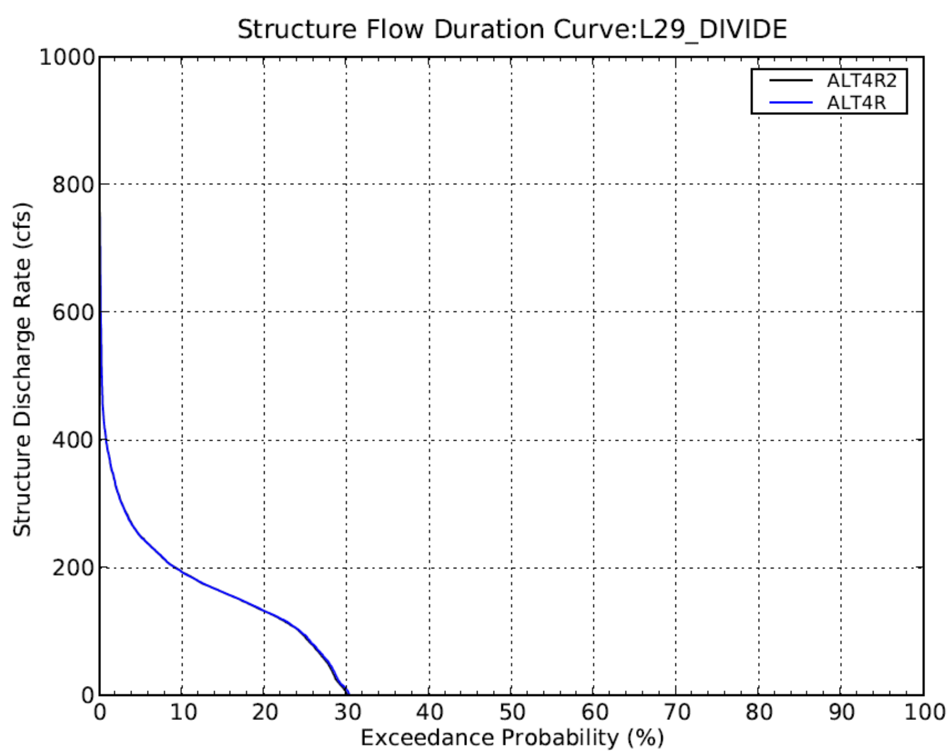


Figure 105B: Flow duration curve for L-29 Canal divide structure

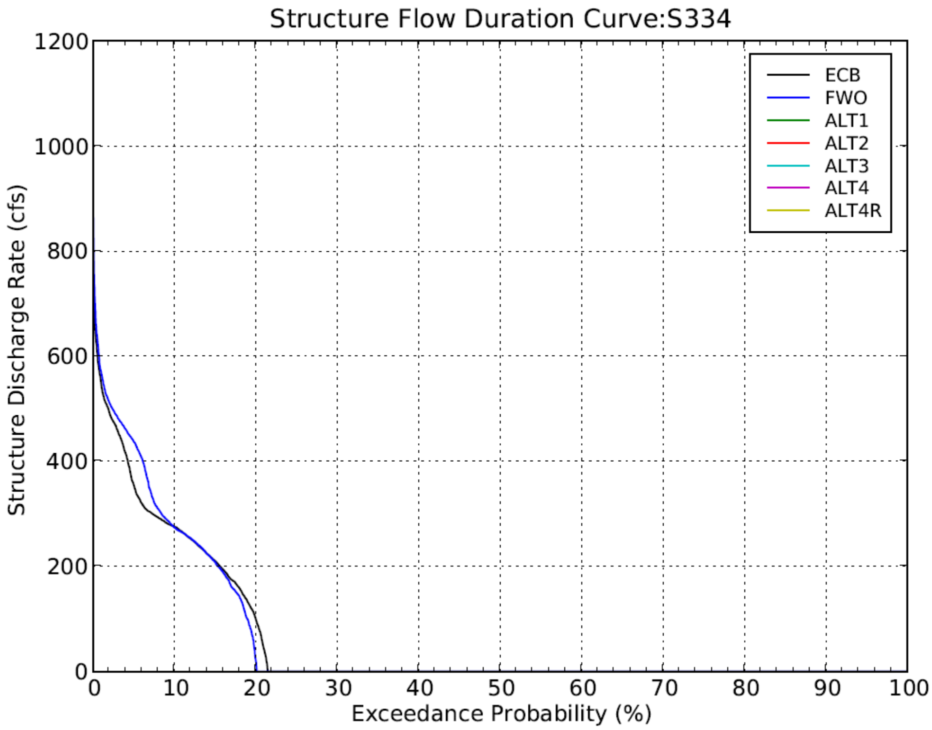


Figure 106A: Flow duration curve for L-29 Canal outlet structure S-334 to SDCS

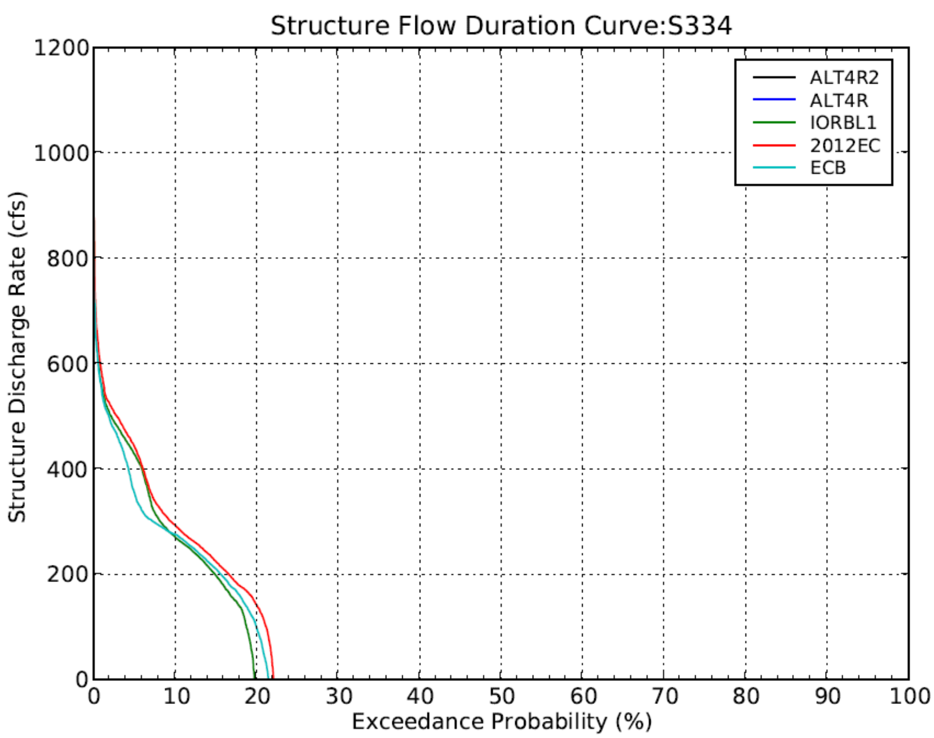


Figure 106B: Flow duration curve for L-29 Canal outlet structure S-334 to SDCS

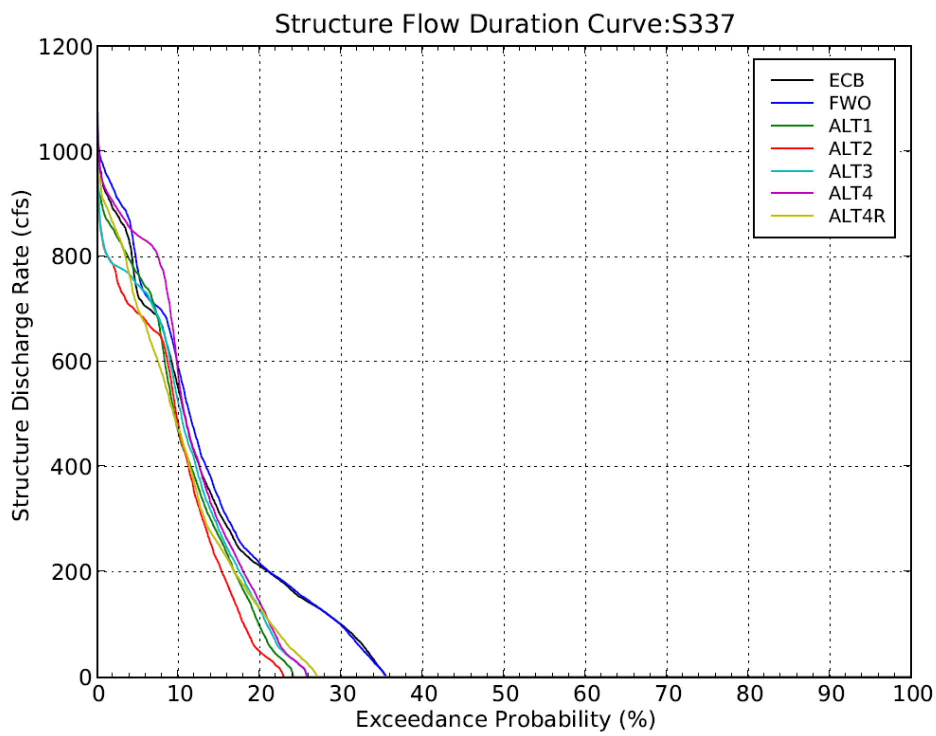


Figure 107A: Flow duration curve for WCA-3B outlet structure S-337 to SDCS

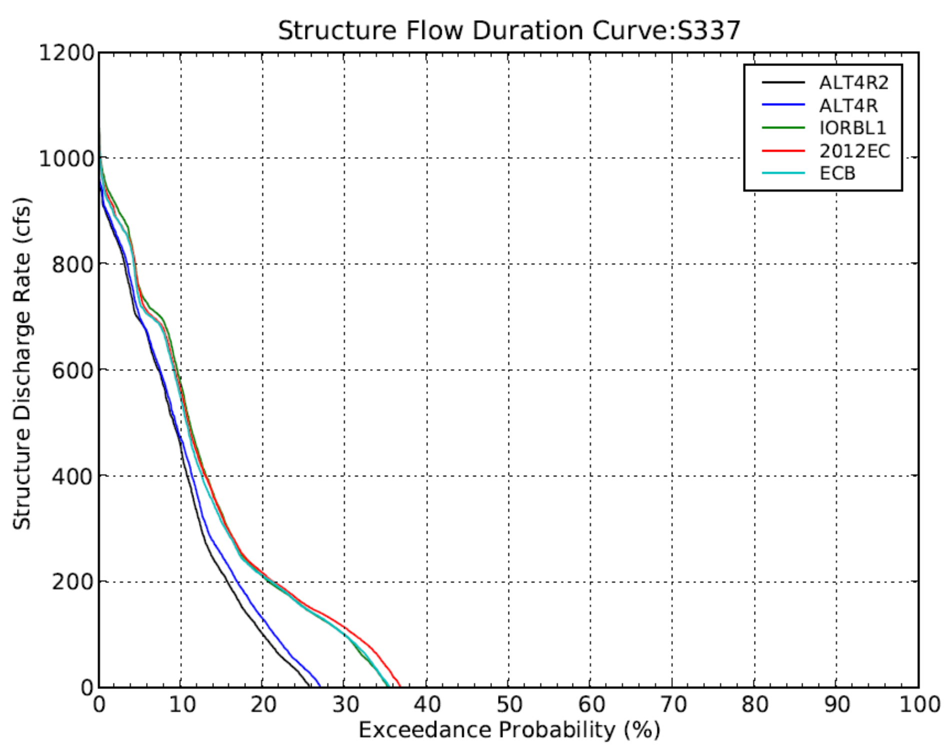


Figure 107B: Flow duration curve for WCA-3B outlet structure S-337 to SDCS

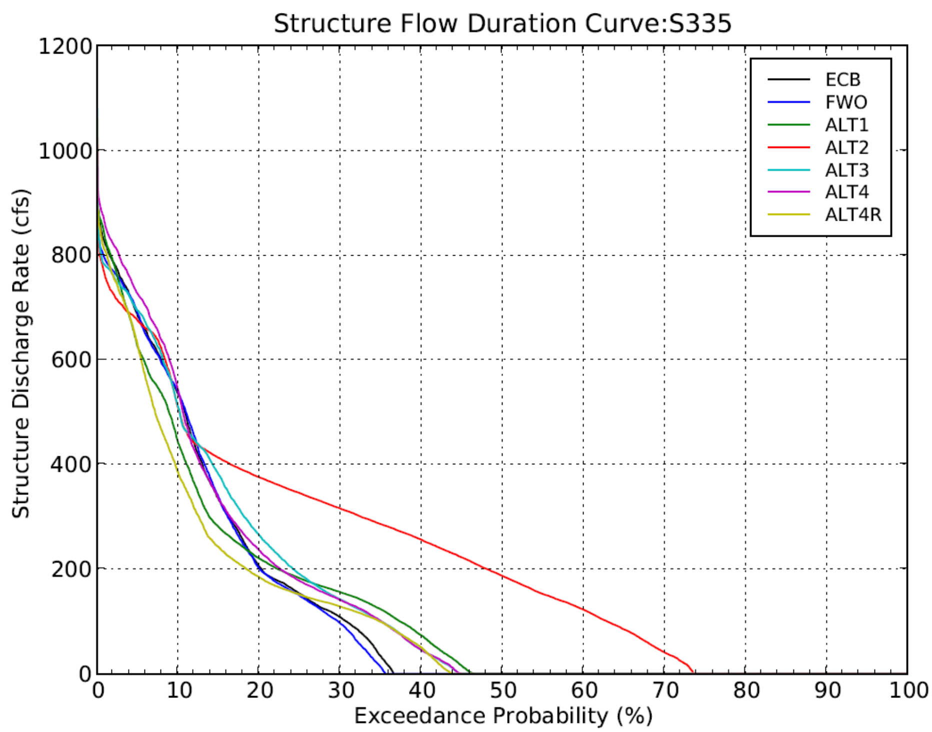


Figure 108A: Flow duration curve for SDCS L-30 Canal outlet structure S-335

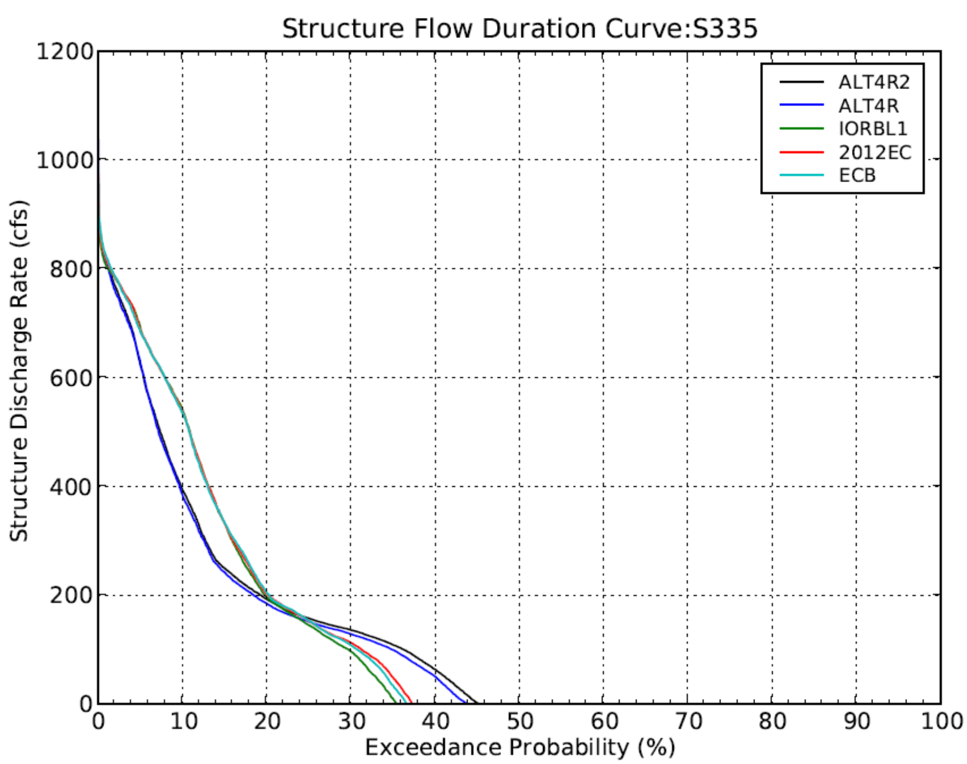


Figure 108B: Flow duration curve for SDCS L-30 Canal outlet structure S-335

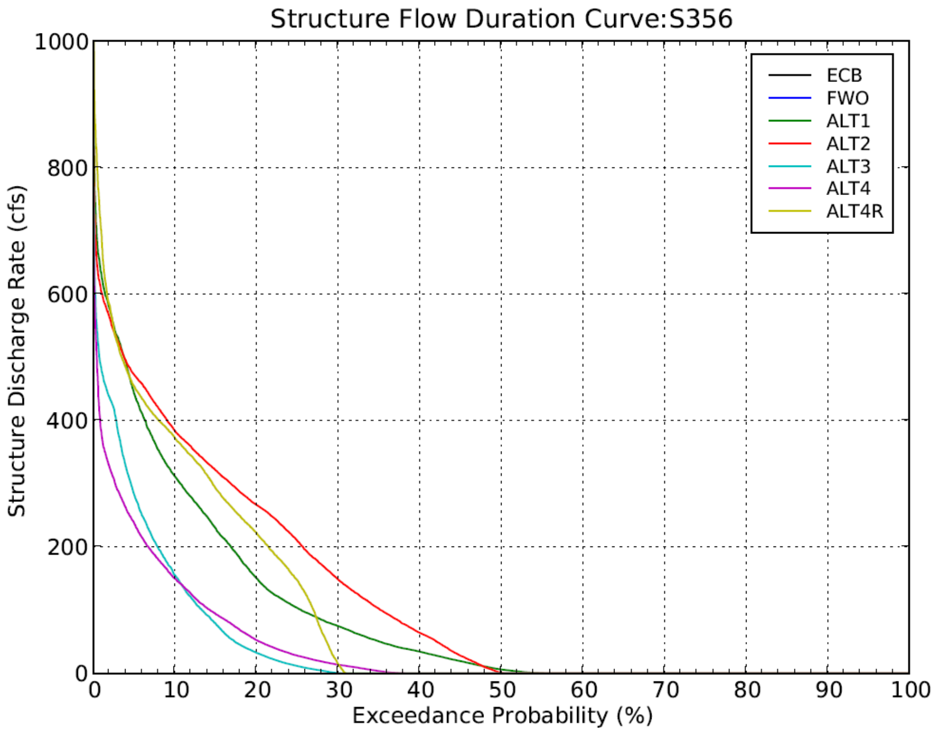


Figure 109A: Flow duration curve for S-356 NESRS seepage return pump station

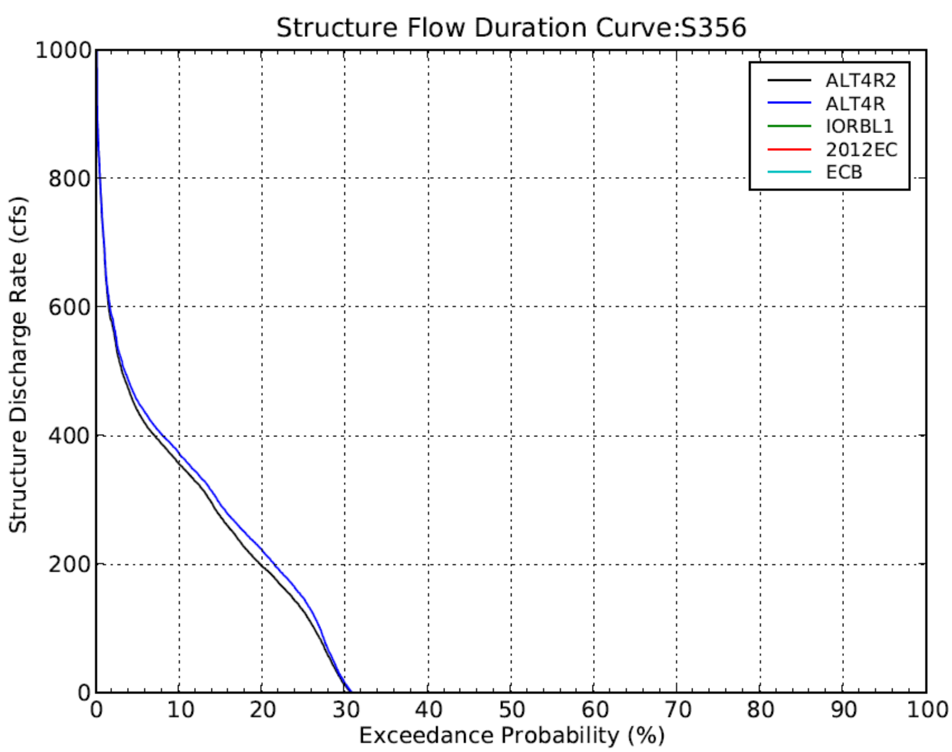


Figure 109B: Flow duration curve for S-356 NESRS seepage return pump station

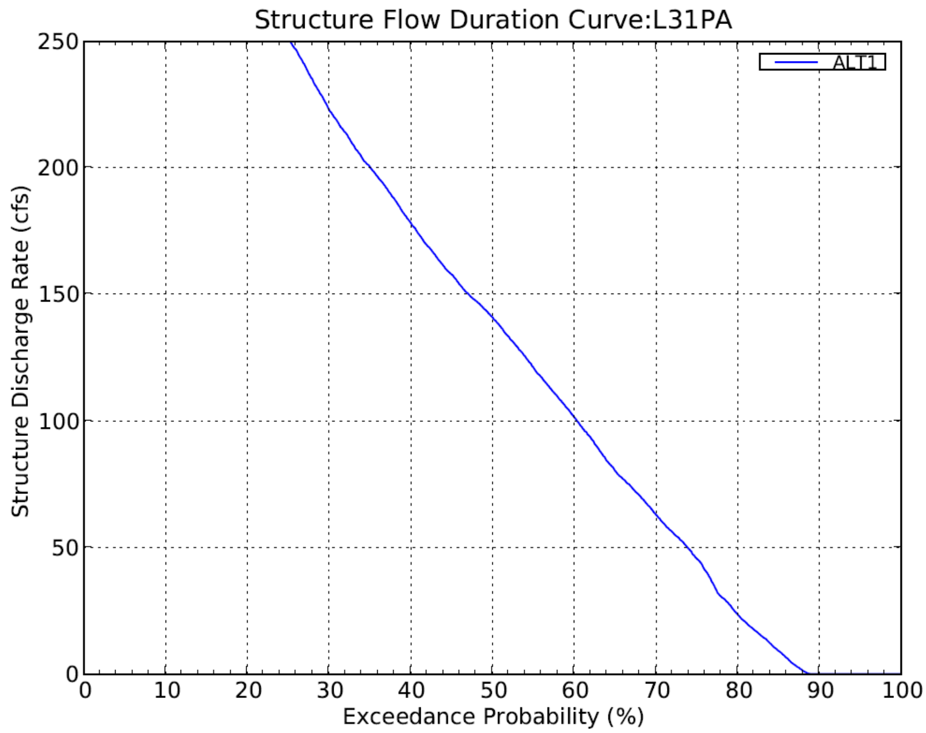


Figure 110A: Flow duration curve for L-31N northern NESRS seepage return pump station L31PA

Note: Structure L-31PA is not included in the simulations displayed in the part B graphics.

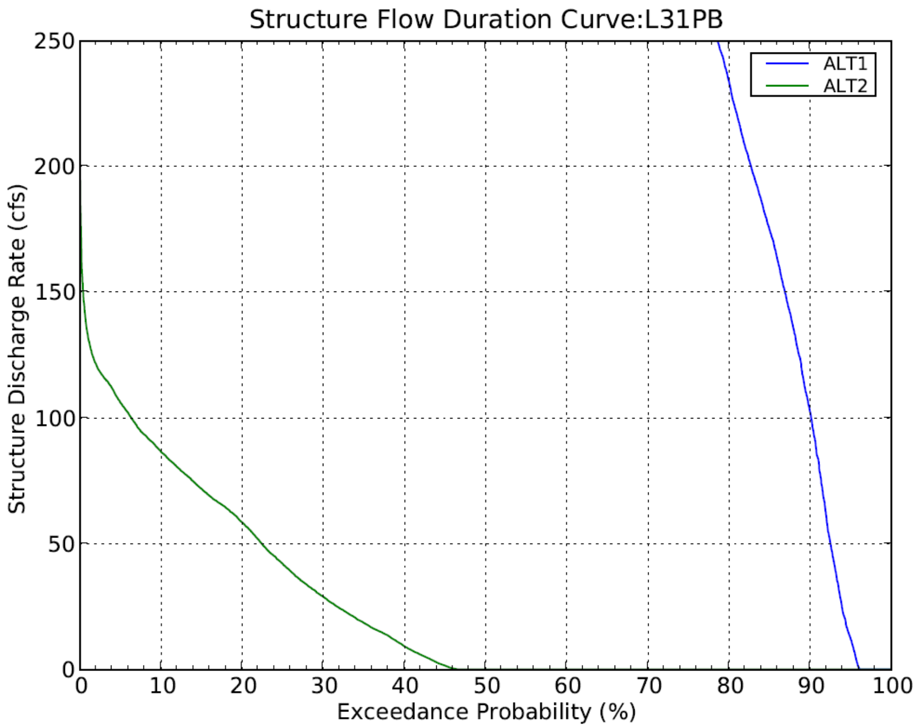


Figure 111A: Flow duration curve for L-31N northern NESRS seepage return pump station L31PB

Note: Structure L-31PB is not included in the simulations displayed in the part B graphics.

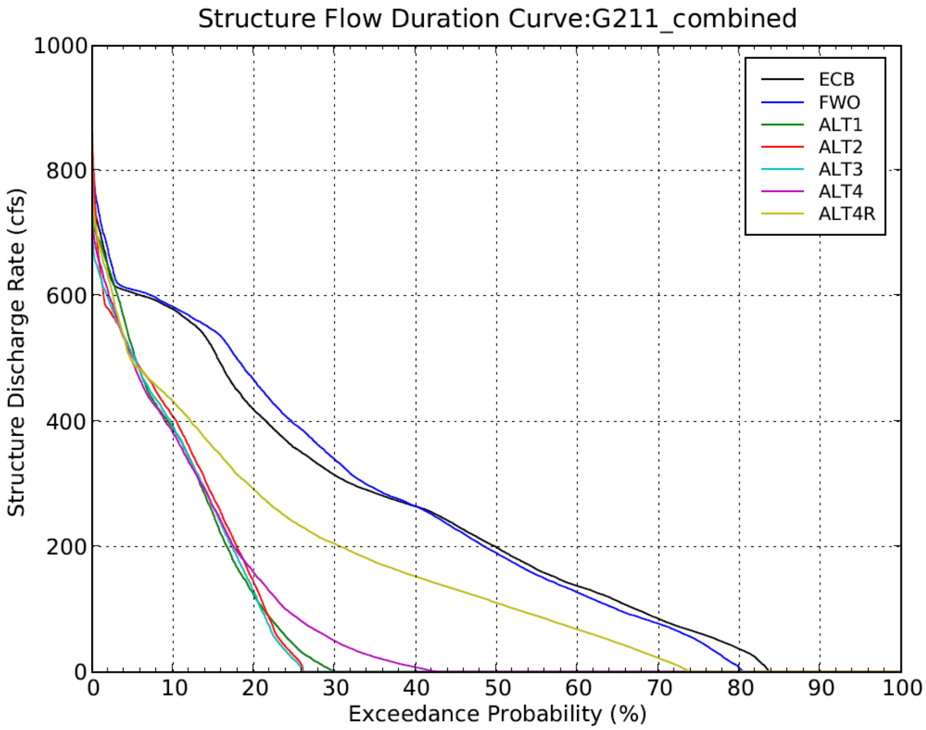


Figure 112A: Flow duration curve for SDCS L-31N Canal outlet structure G-211

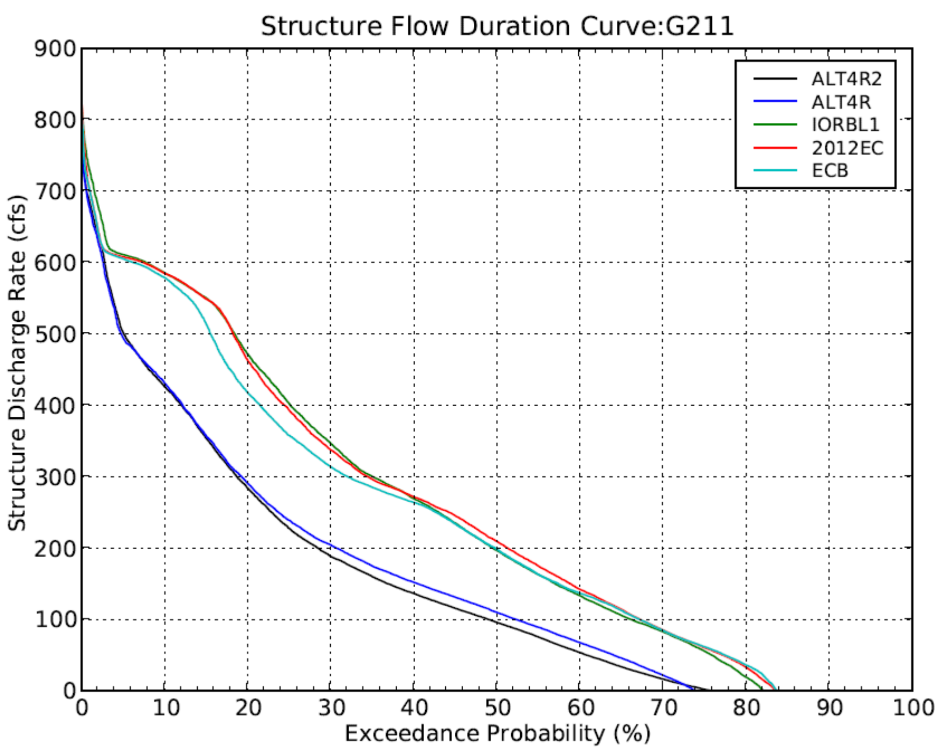


Figure 112B: Flow duration curve for SDCS L-31N Canal outlet structure G-211

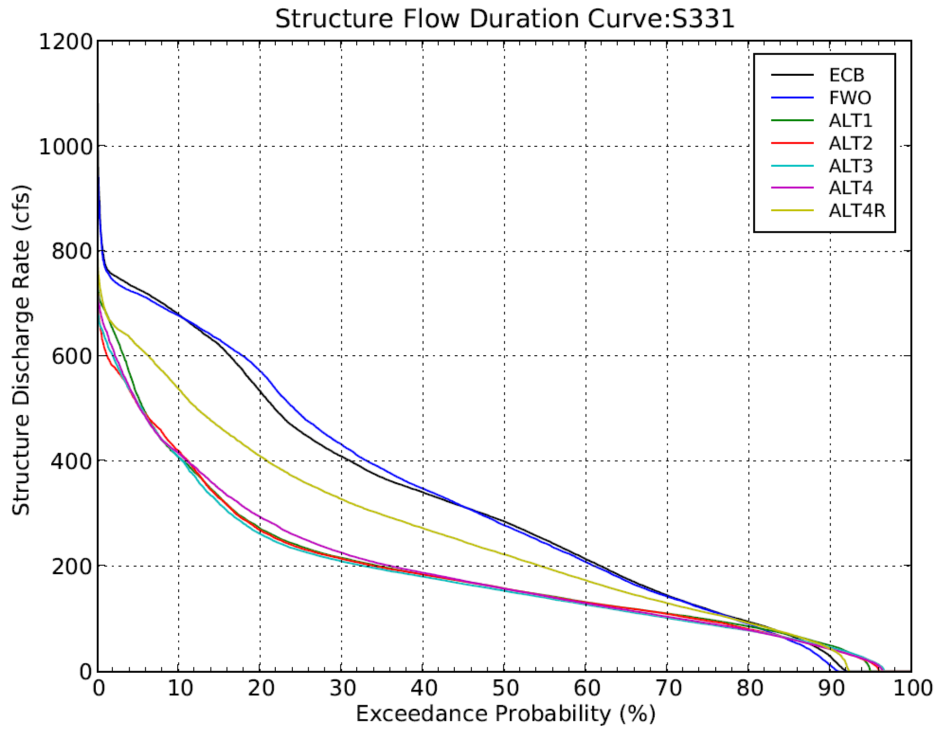


Figure 113A: Flow duration curve for SDCS L-31N Canal outlet structure S-331

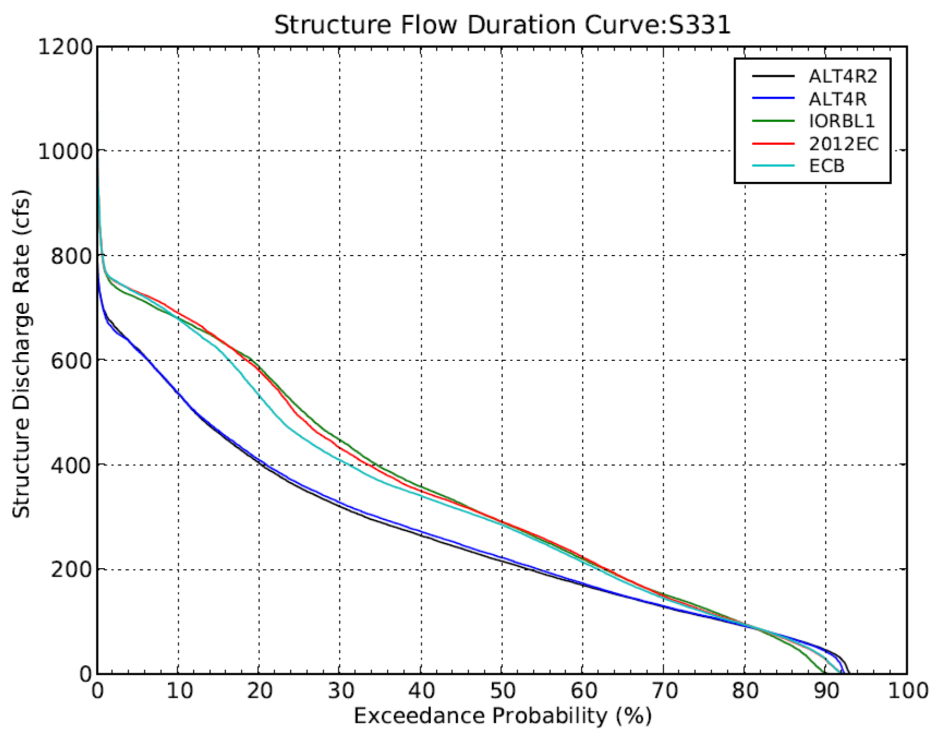


Figure 113B: Flow duration curve for SDCS L-31N Canal outlet structure S-331

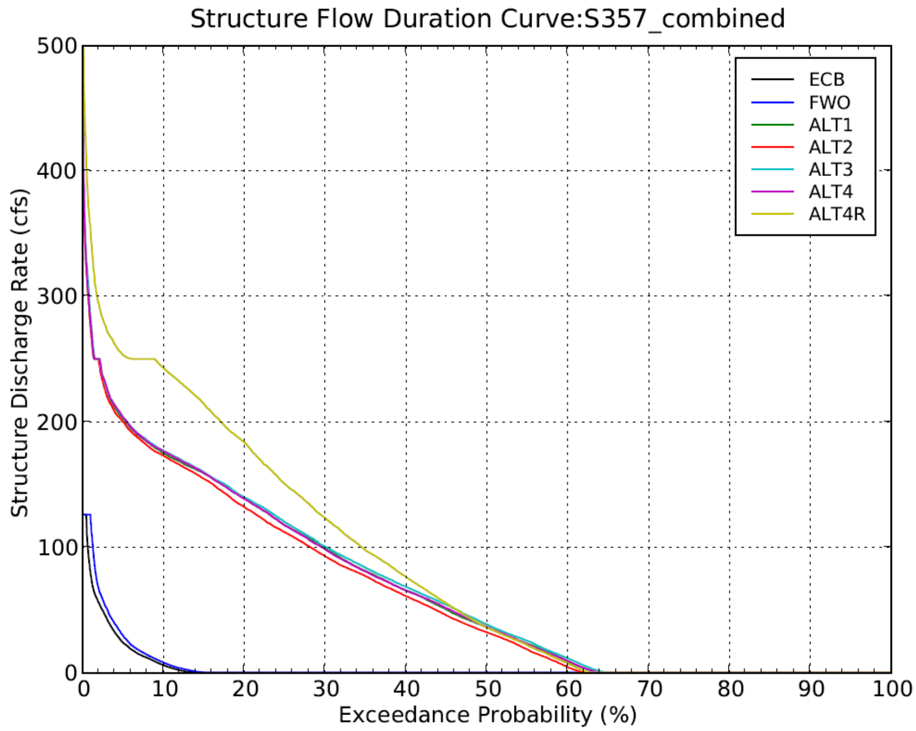


Figure 114A: Combined flow duration curve for 8.5 SMA seepage collection canal outlet structure S-357

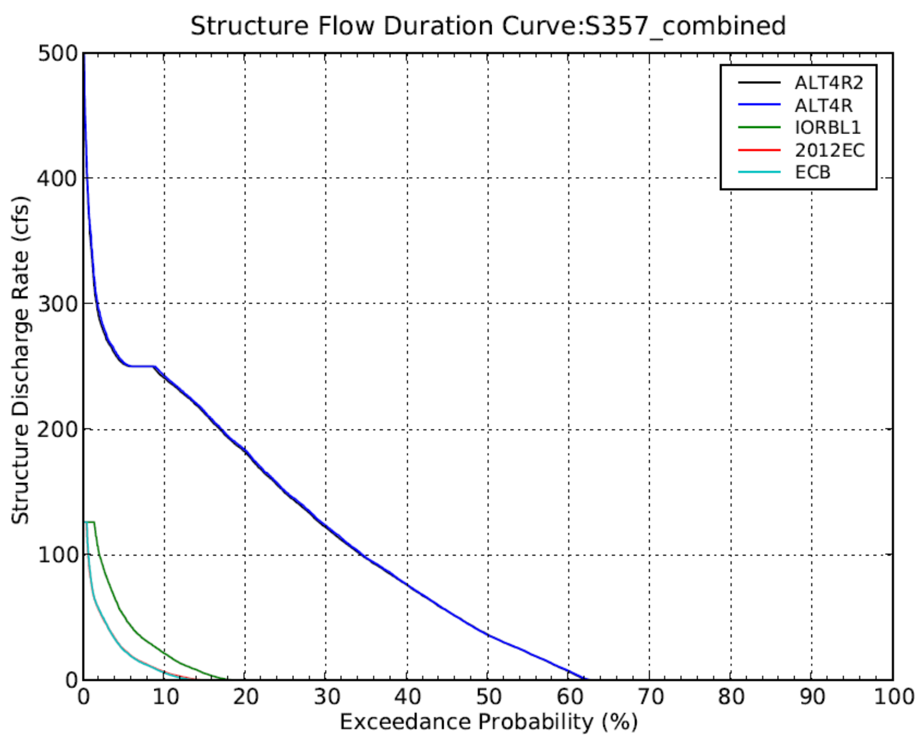


Figure 114B: Combined flow duration curve for 8.5 SMA seepage collection canal outlet structure S-357

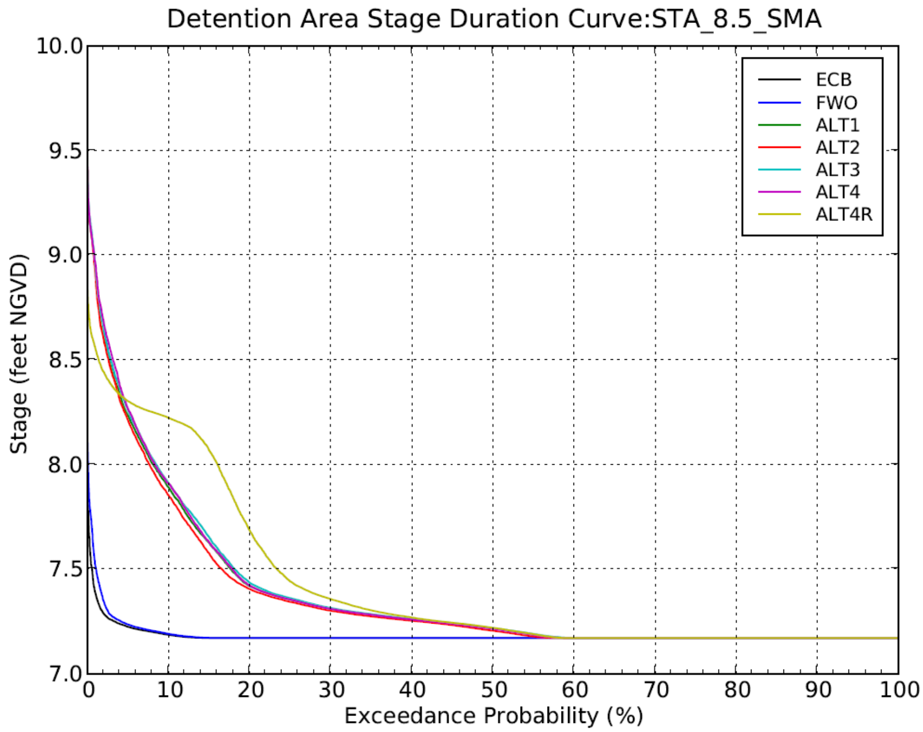


Figure 115A: Stage duration curve for 8.5 SMA detention cell

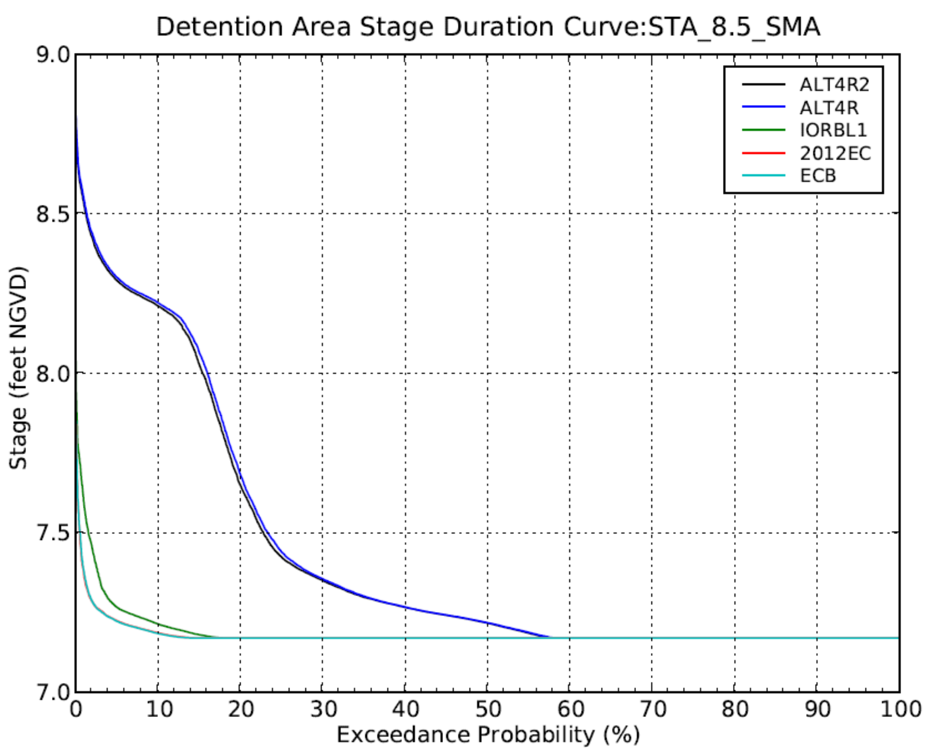


Figure 115B: Stage duration curve for 8.5 SMA detention cell

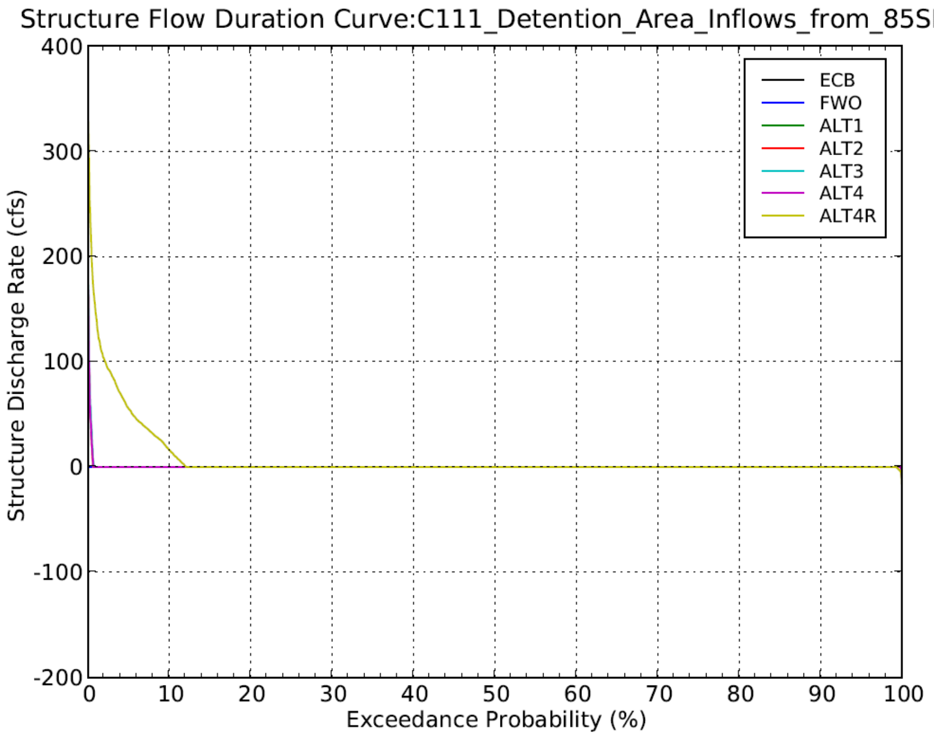


Figure 116A: Combined flow duration curve for 8.5 SMA detention cell outlet structures S-360W and S-360E

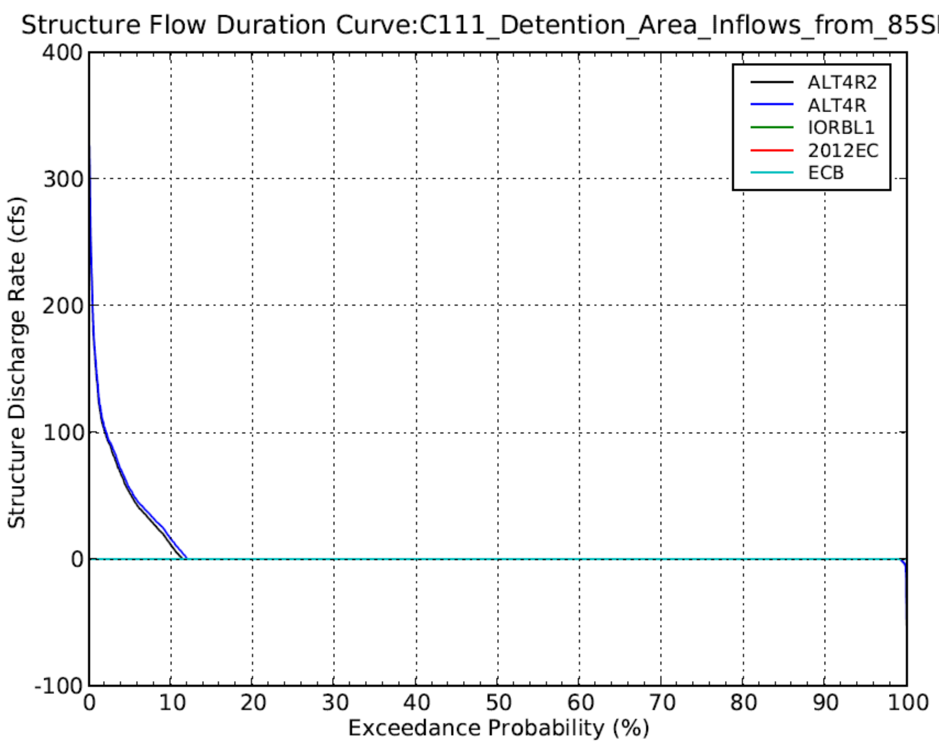


Figure 116B: Combined flow duration curve for 8.5 SMA detention cell outlet structures S-360W and S-360E

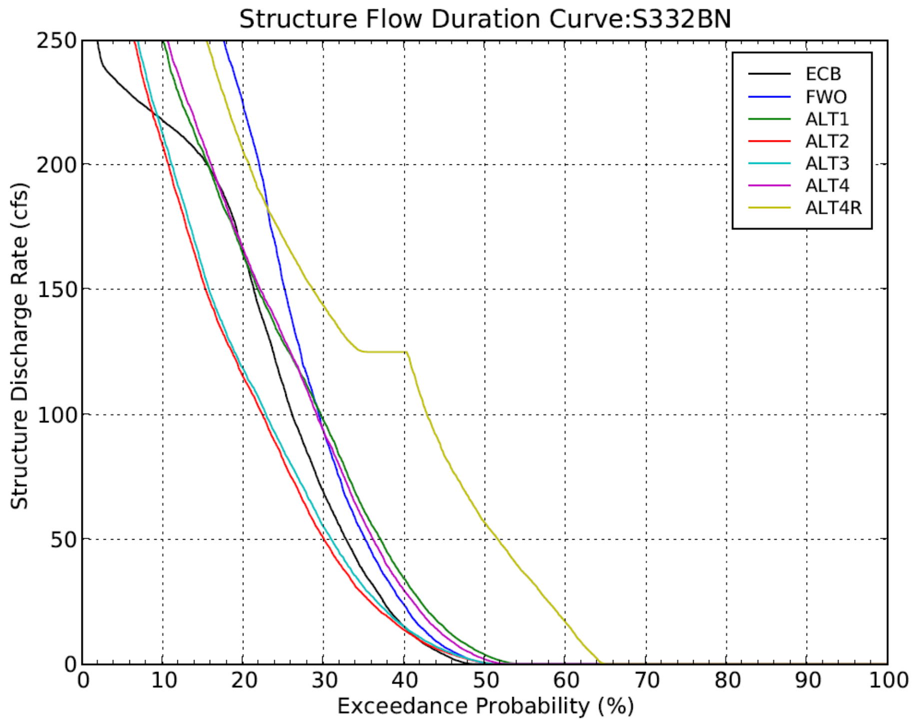


Figure 117A: Flow duration curve for SDCS L-31N Canal outlet structure S-332B to C-111 NDA

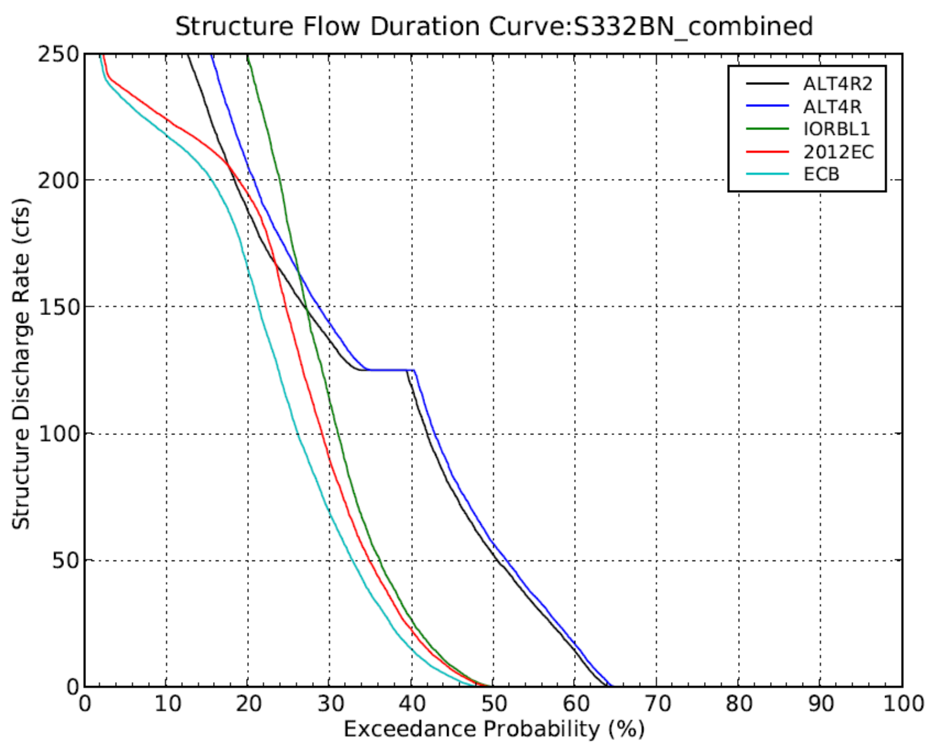


Figure 117B: Flow duration curve for SDCS L-31N Canal outlet structure S-332B to C-111 NDA

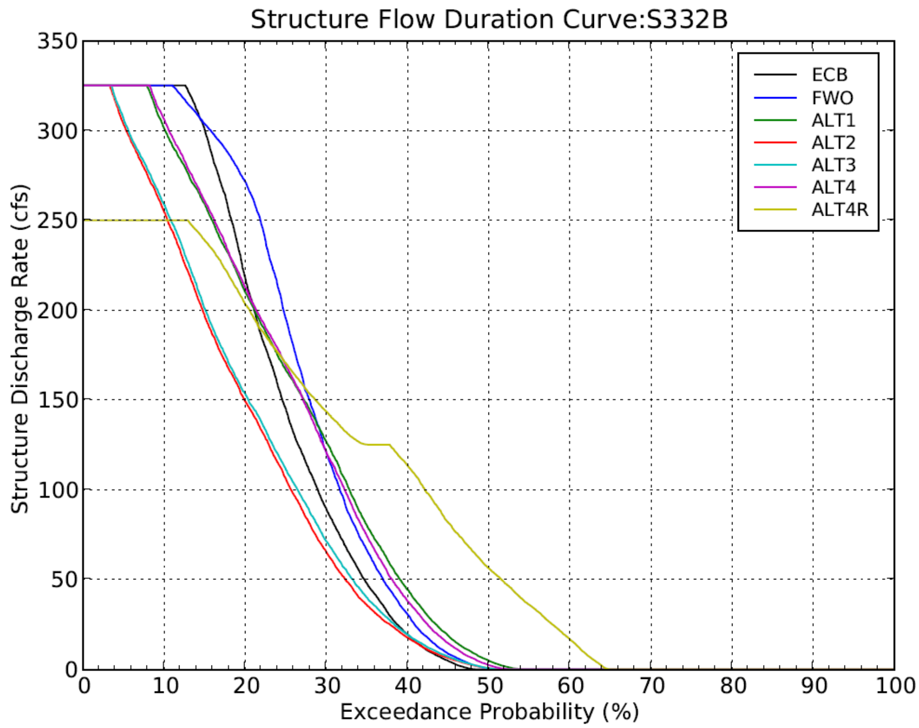


Figure 118A: Flow duration curve for SDCS L-31N Canal outlet structure S-332B to C-111 SDA

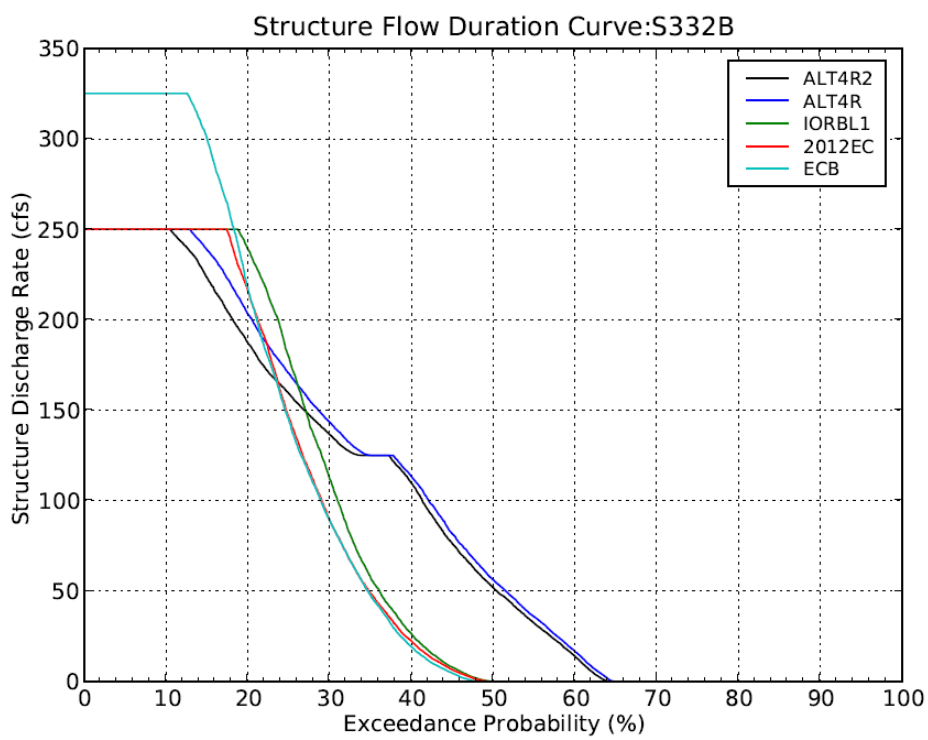


Figure 118B: Flow duration curve for SDCS L-31N Canal outlet structure S-332B to C-111 SDA

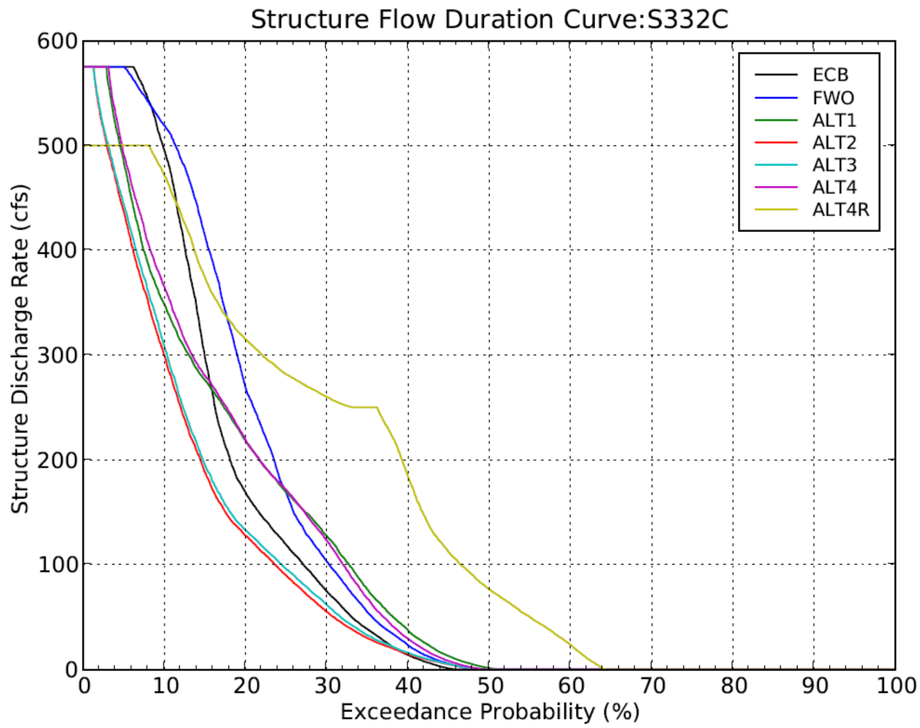


Figure 119A: Flow duration curve for SDCS L-31N Canal outlet structure S-332C to C-111 SDA

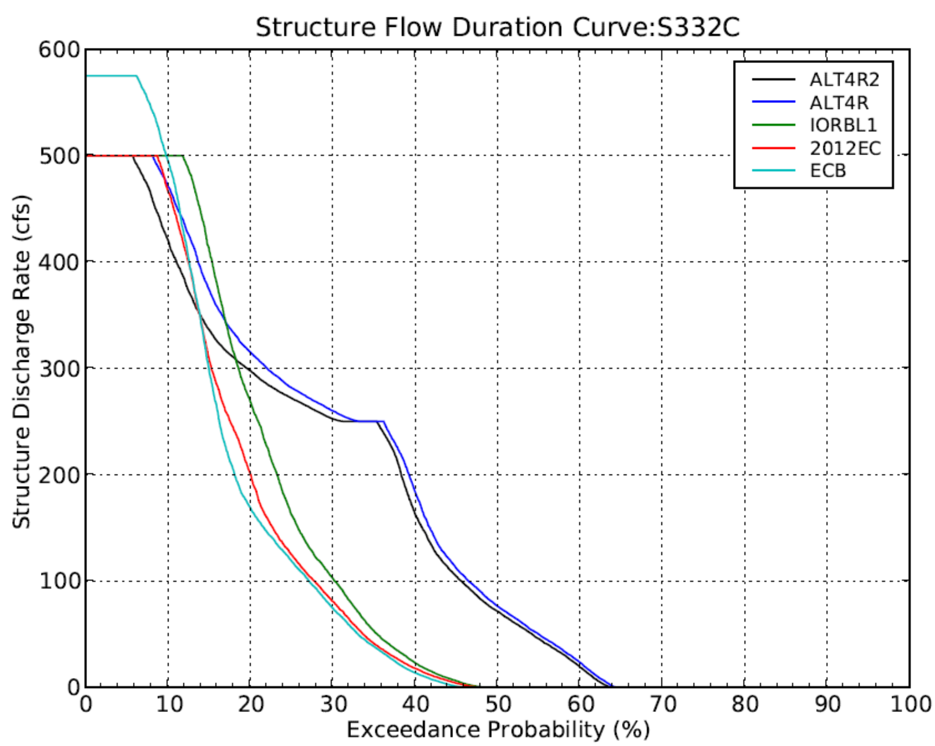


Figure 119B: Flow duration curve for SDCS L-31N Canal outlet structure S-332C to C-111 SDA

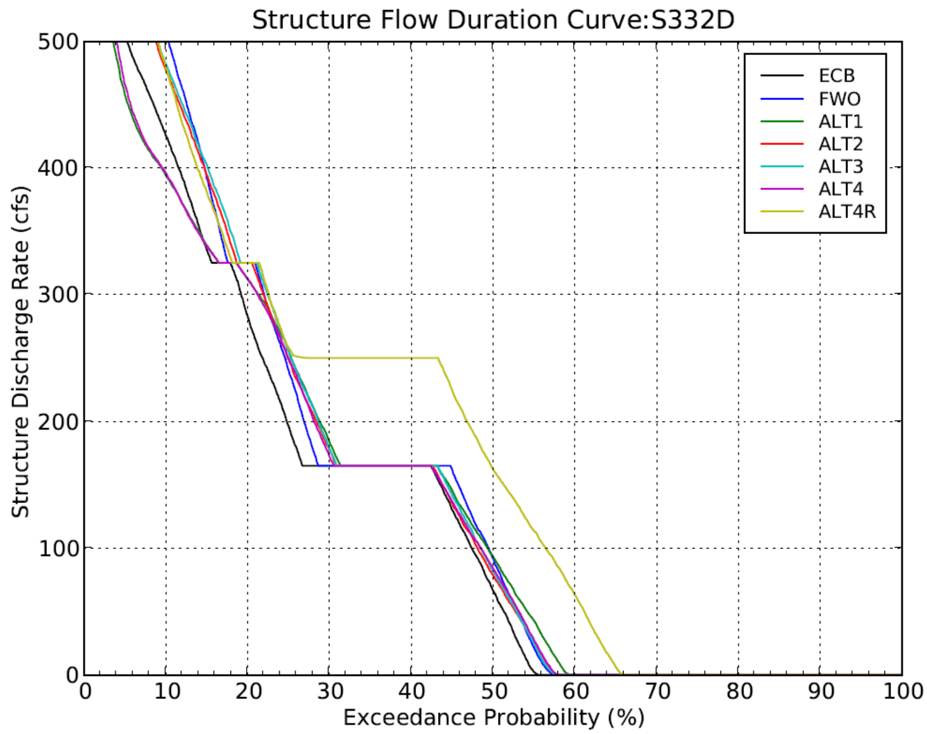


Figure 120A: Flow duration curve for SDCS L-31N Canal outlet structure S-332D to Taylor Slough

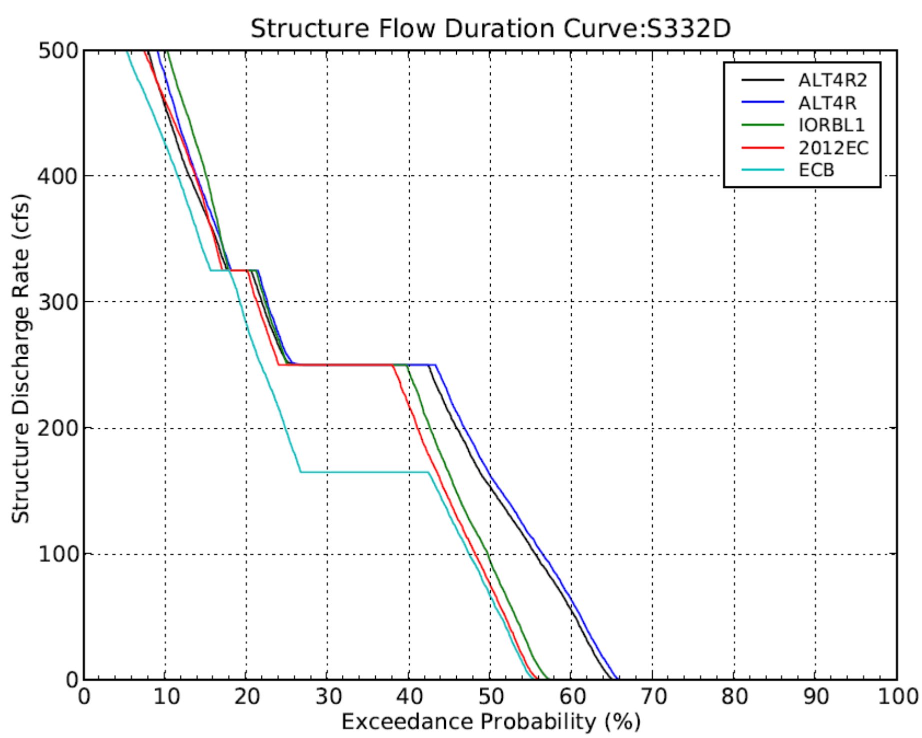


Figure 120B: Flow duration curve for SDCS L-31N Canal outlet structure S-332D to Taylor Slough

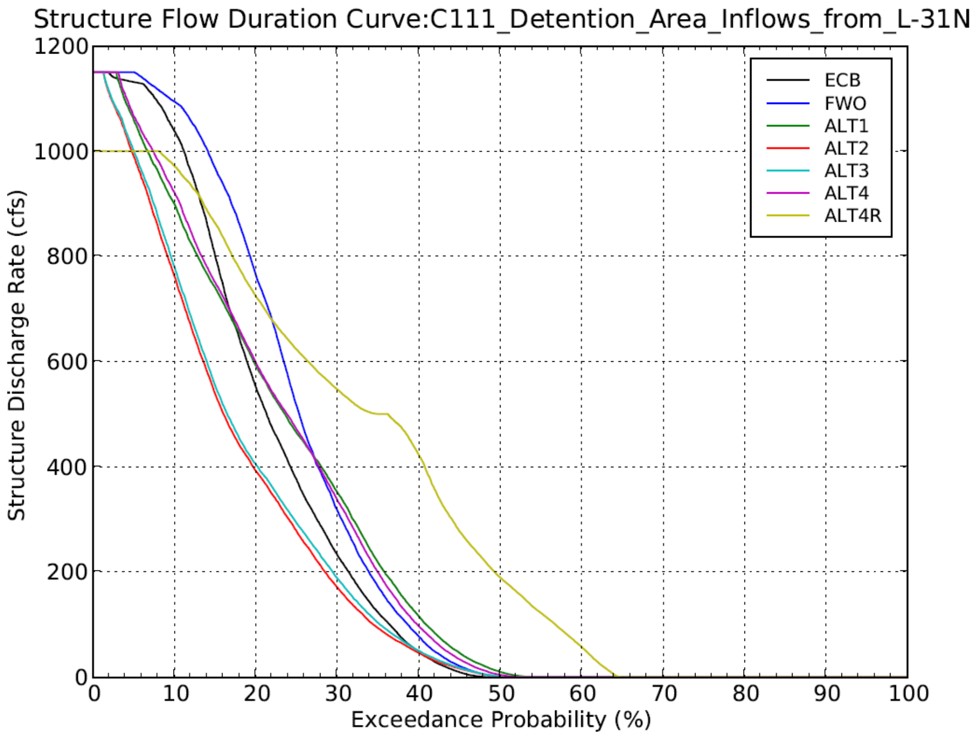


Figure 121A: Combined flow duration curve for SDCS L-31N Canal outlet structures S-332B and S-332C

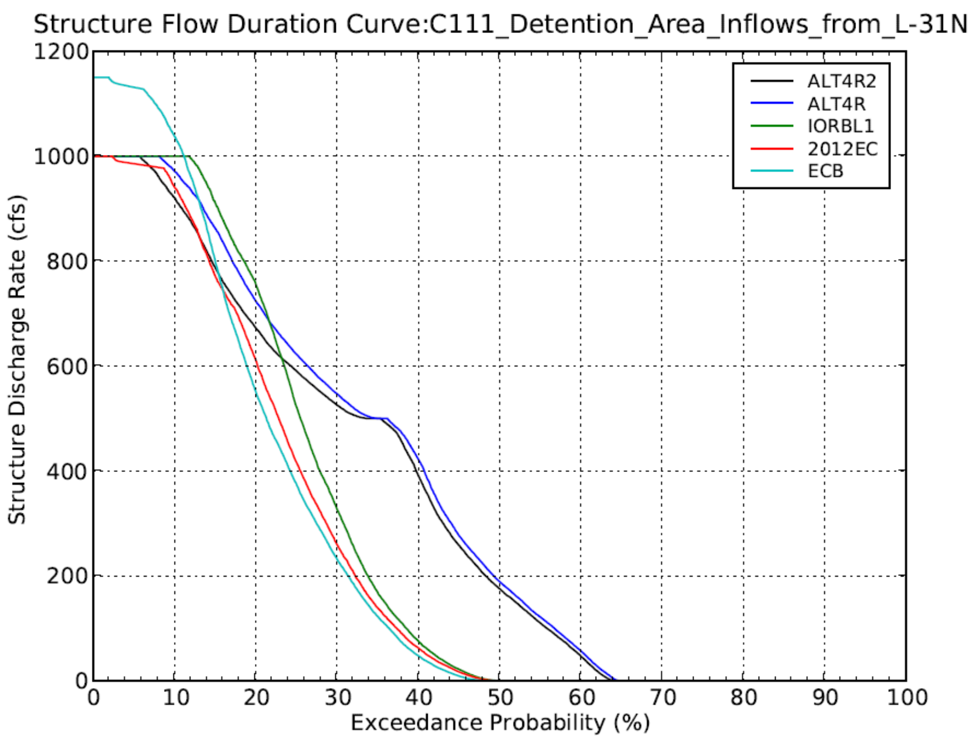


Figure 121B: Combined flow duration curve for SDCS L-31N Canal outlet structures S-332B and S-332C

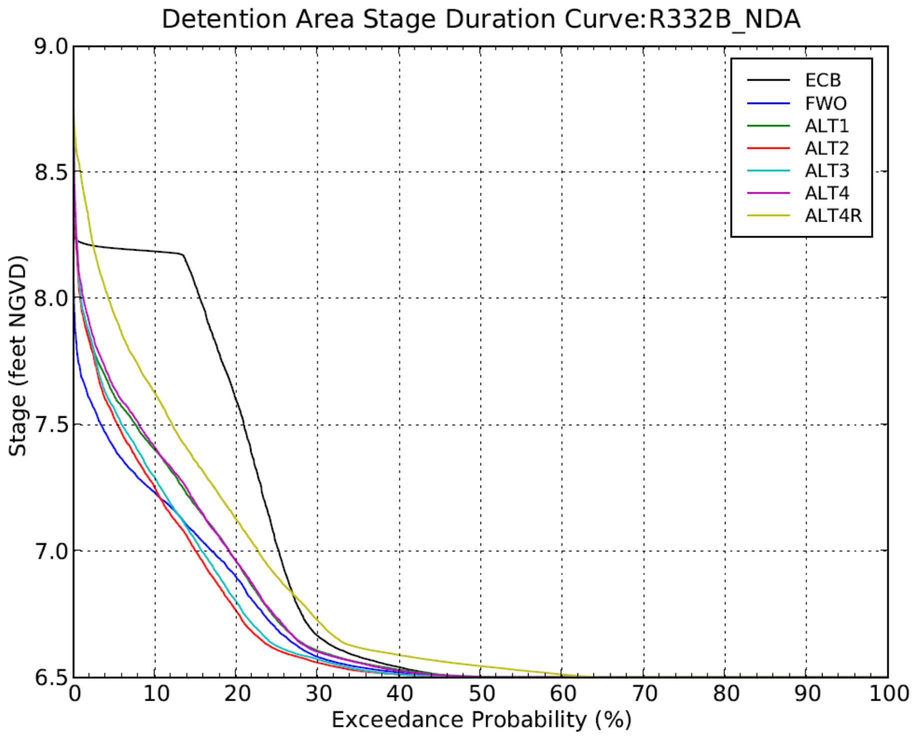


Figure 122A: Stage duration curve for C-111 North Detention Area

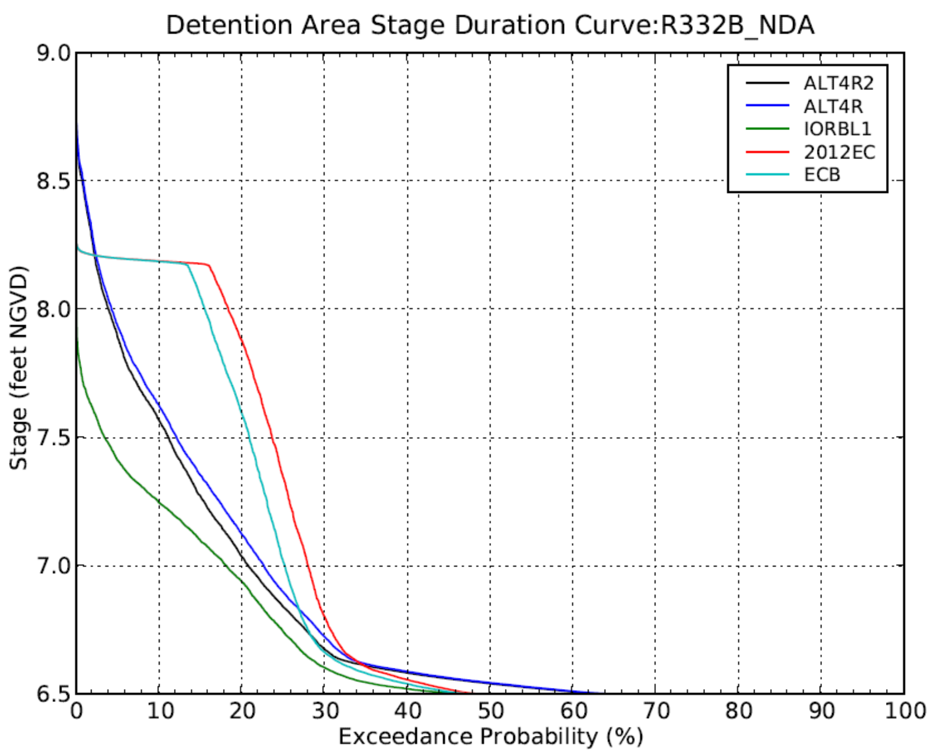


Figure 122B: Stage duration curve for C-111 North Detention Area

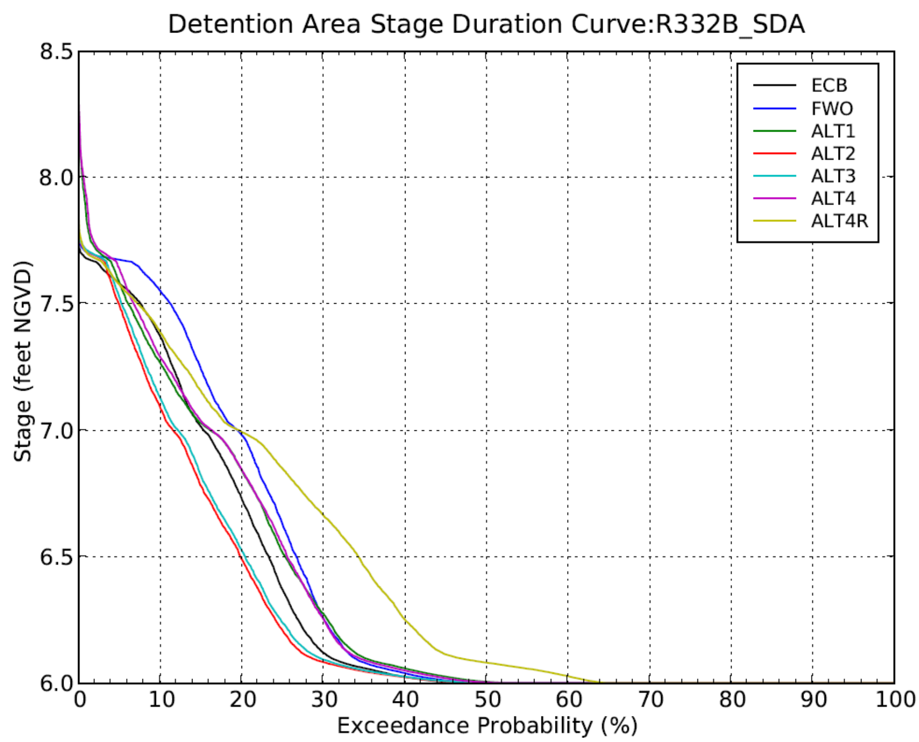


Figure 123A: Stage duration curve for C-111 South Detention Area

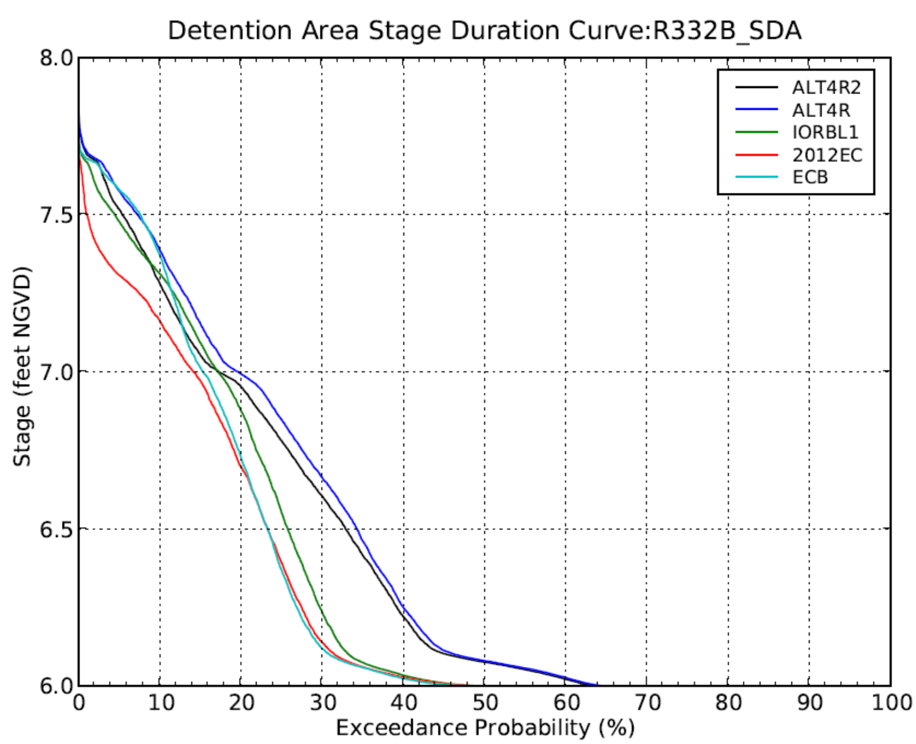


Figure 123B: Stage duration curve for C-111 South Detention Area

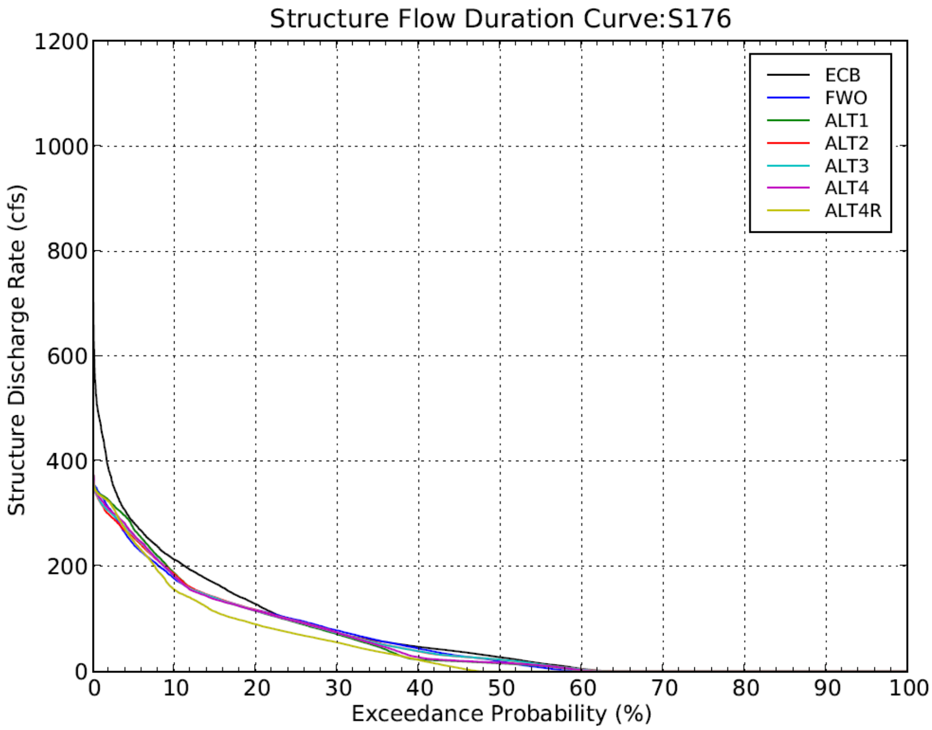


Figure 124A: Flow duration curve for SDCS L-31N Canal outlet structure S-176

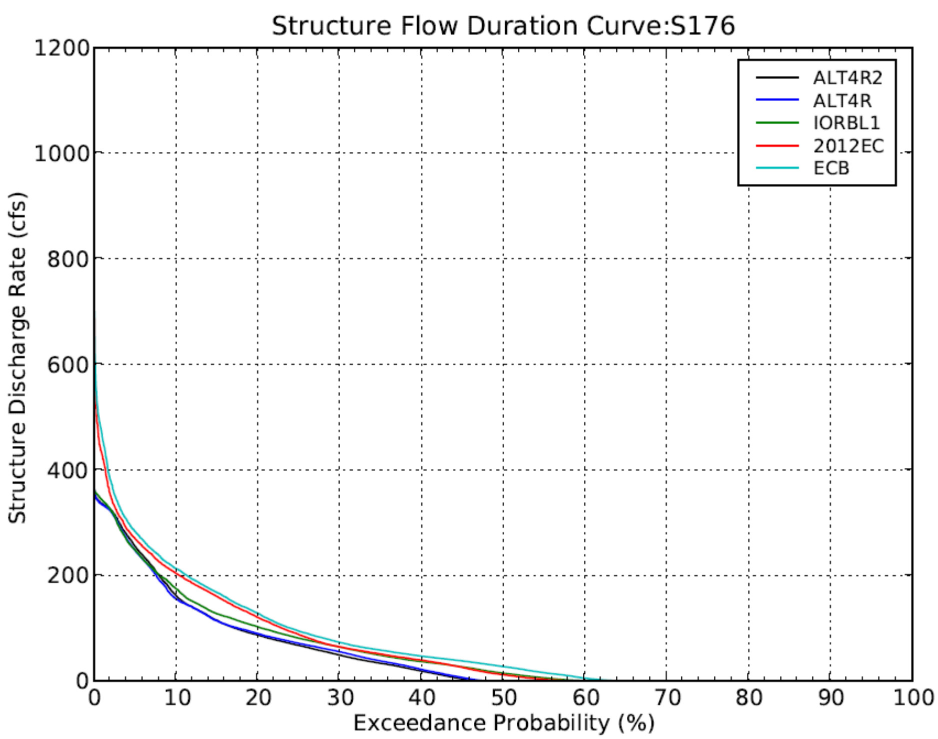


Figure 124B: Flow duration curve for SDCS L-31N Canal outlet structure S-176

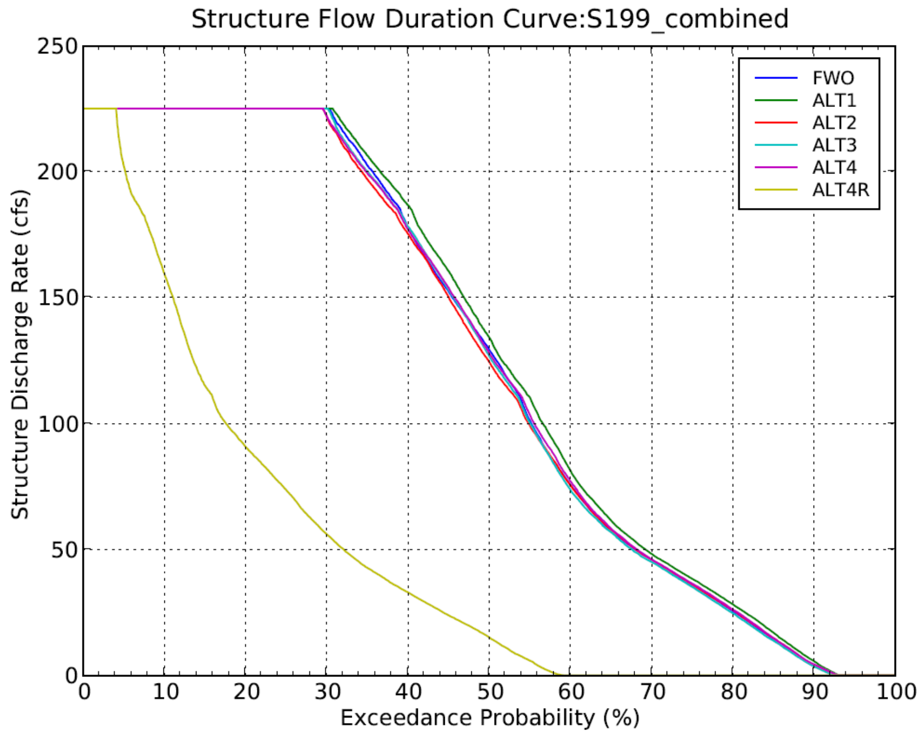


Figure 125A: Combined flow duration curve for CERP C-111 Spreader Canal Western PIR pump station S-199

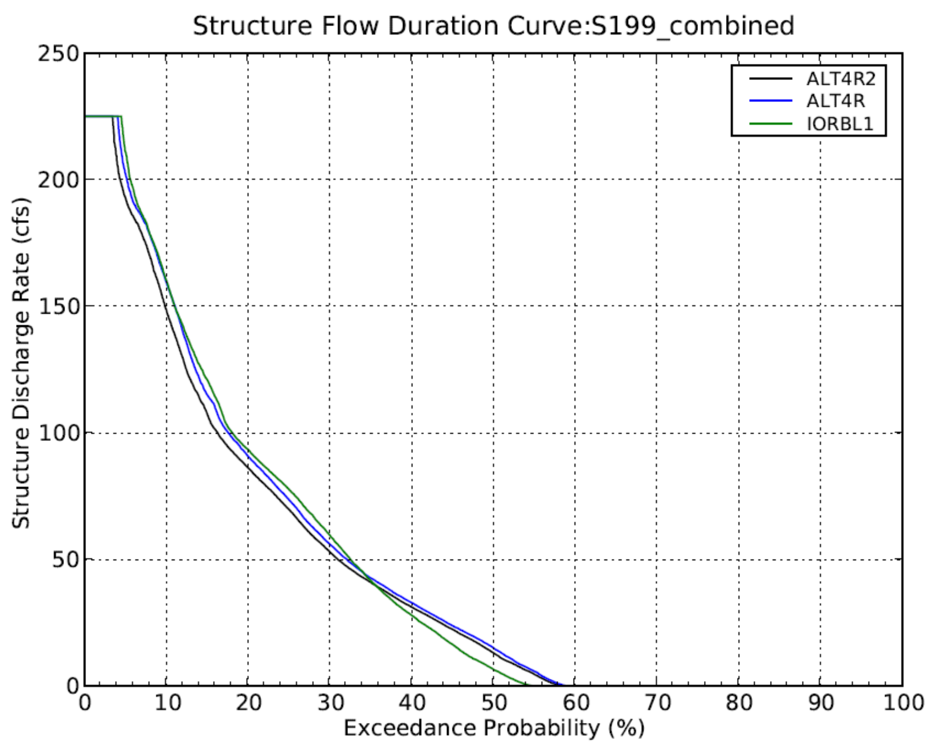


Figure 125B: Combined flow duration curve for CERP C-111 Spreader Canal Western PIR pump station S-199

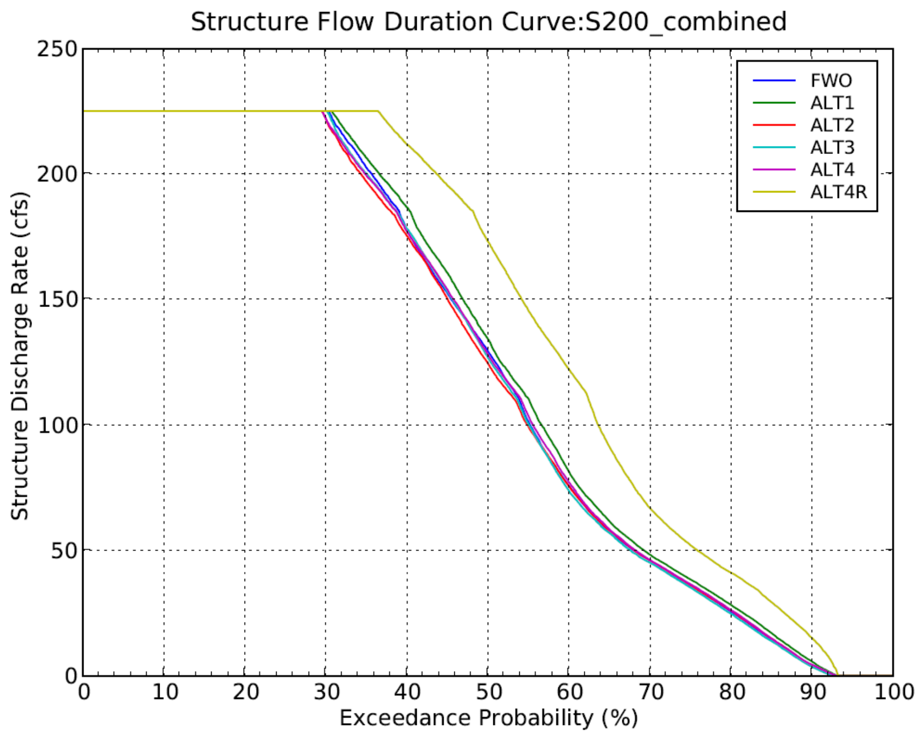


Figure 126A: Combined flow duration curve for CERP C-111 Spreader Canal Western PIR pump station S-200

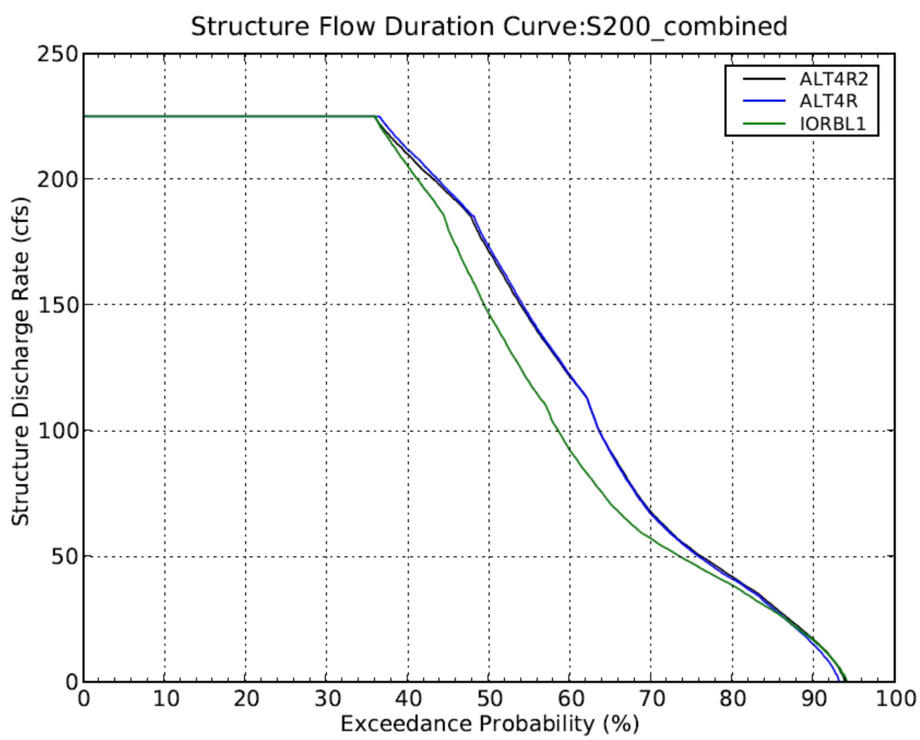


Figure 126B: Combined flow duration curve for CERP C-111 Spreader Canal Western PIR pump station S-200

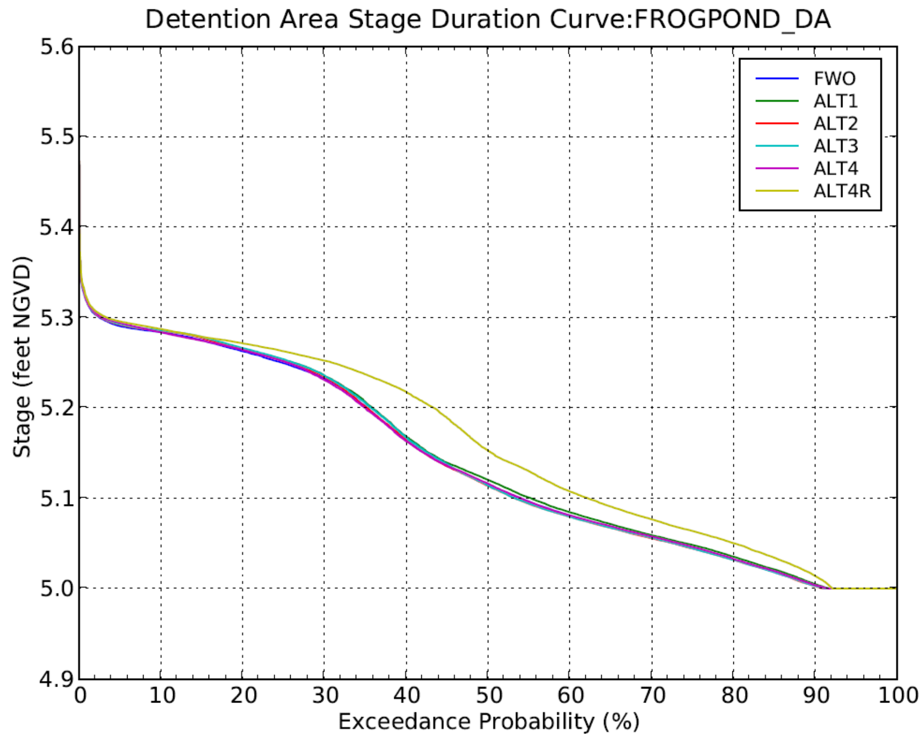


Figure 127A: Stage duration curve for CERP Spreader Canal Western PIR Frog Pond Detention Area

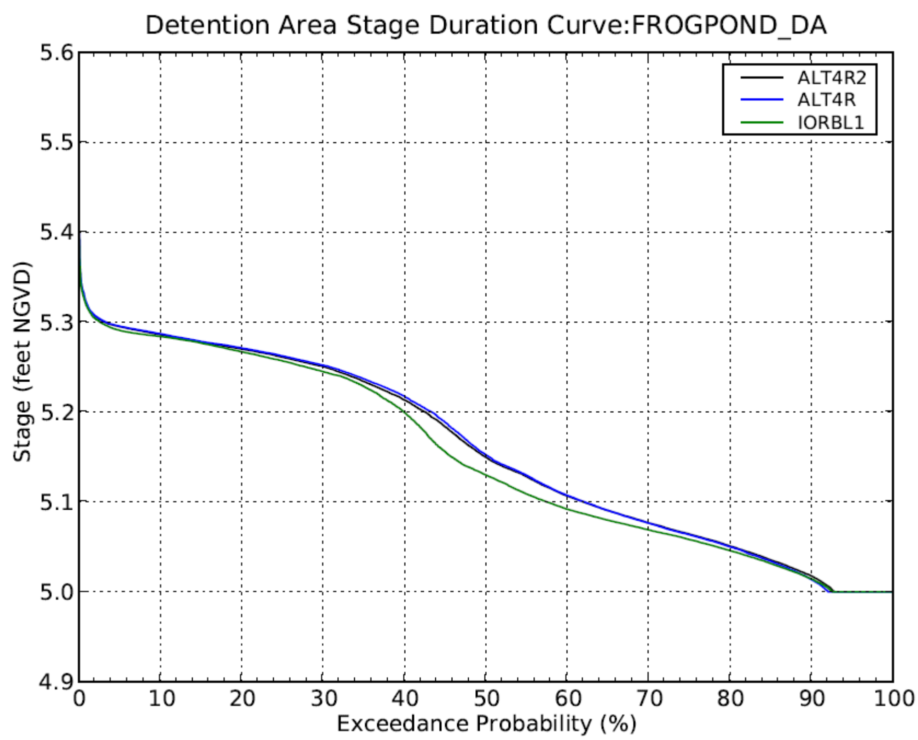


Figure 127B: Stage duration curve for CERP Spreader Canal Western PIR Frog Pond Detention Area

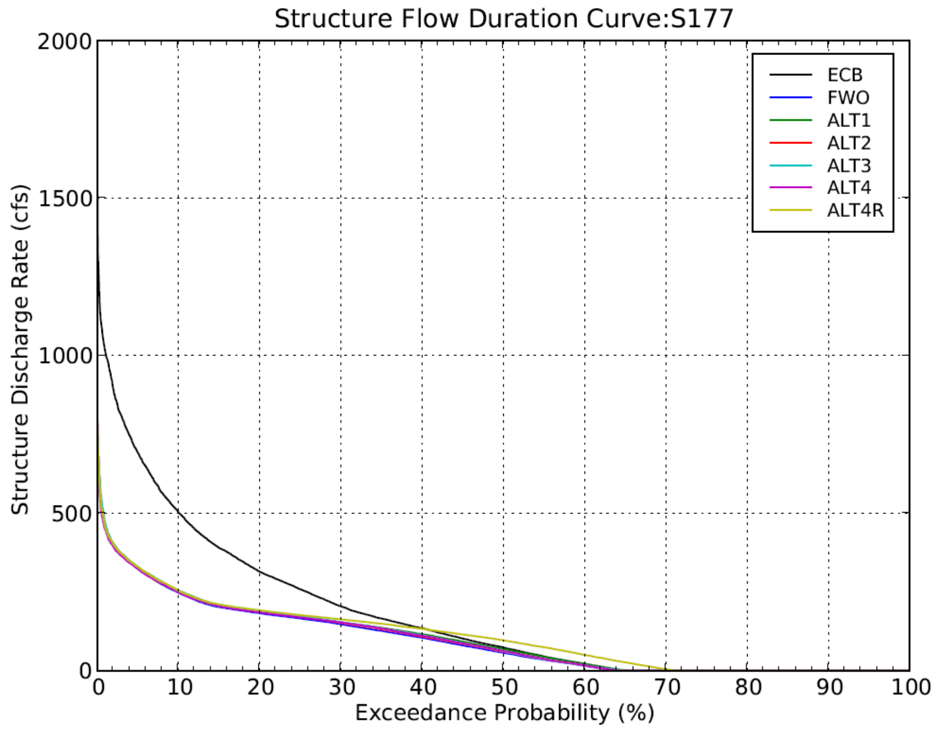


Figure 128A: Flow duration curve for SDCS C-111 Canal outlet structure S-177

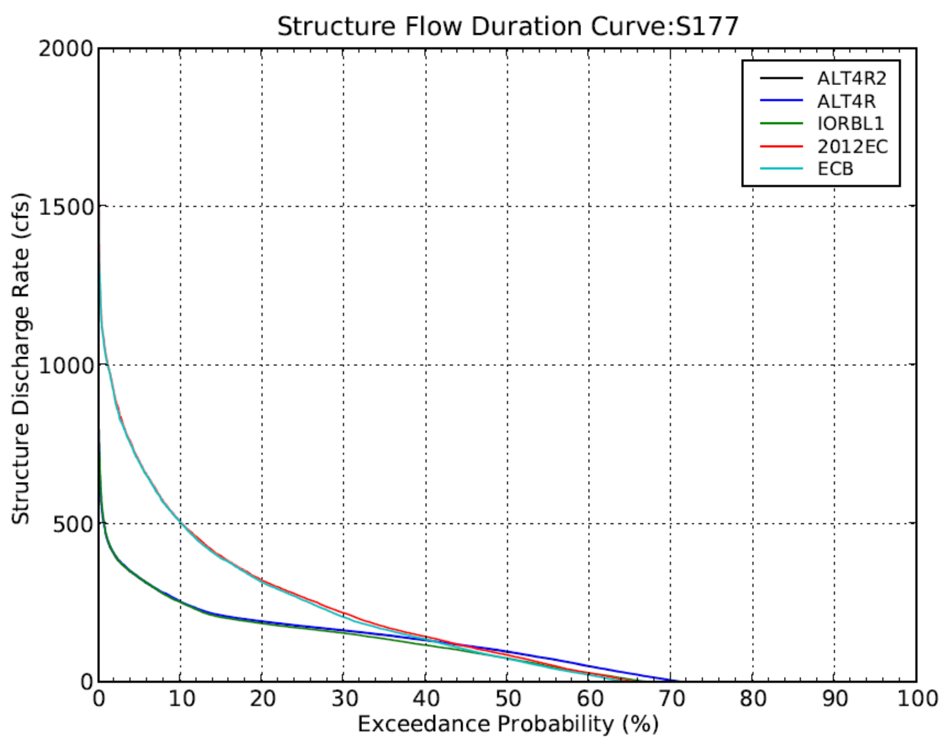


Figure 128B: Flow duration curve for SDCS C-111 Canal outlet structure S-177

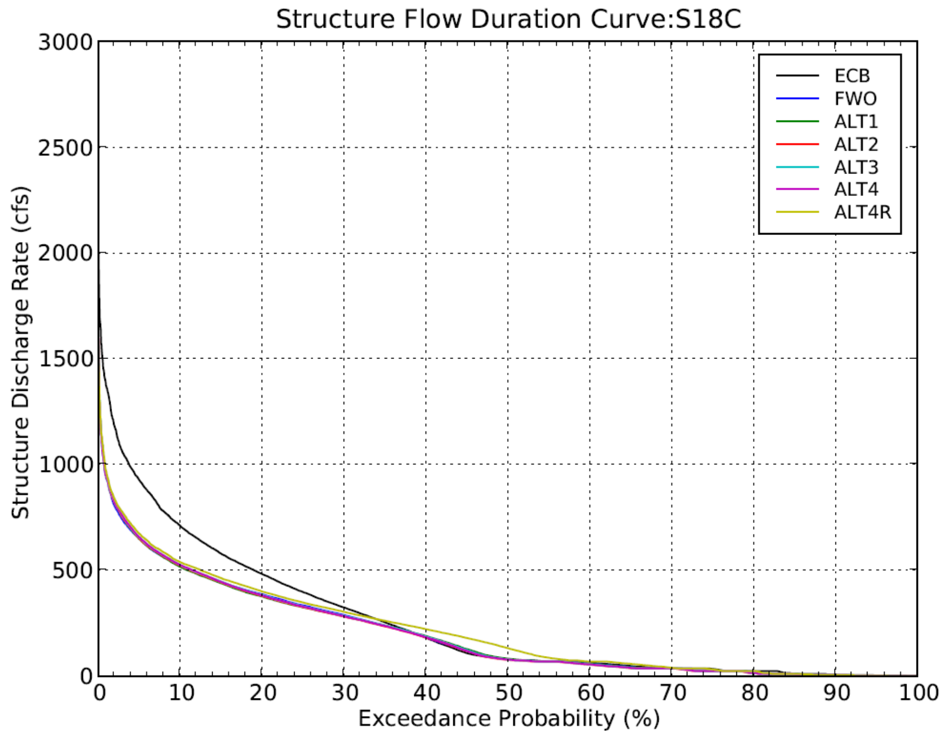


Figure 129A: Flow duration curve for SDCS C-111 Canal outlet structure S-18C

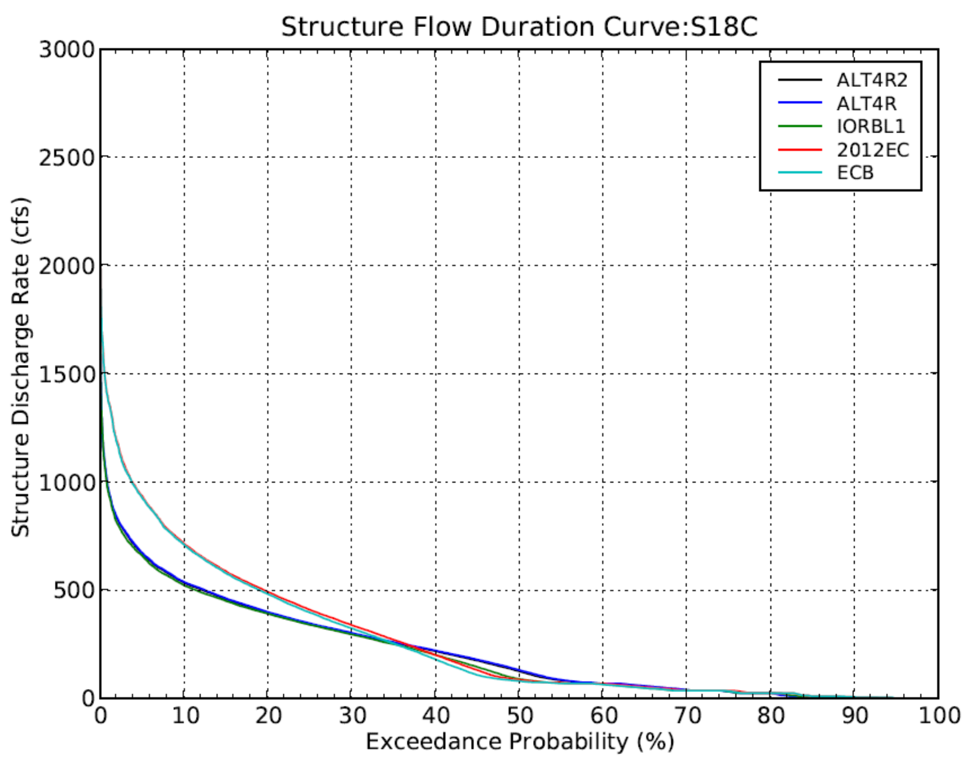


Figure 129B: Flow duration curve for SDCS C-111 Canal outlet structure S-18C

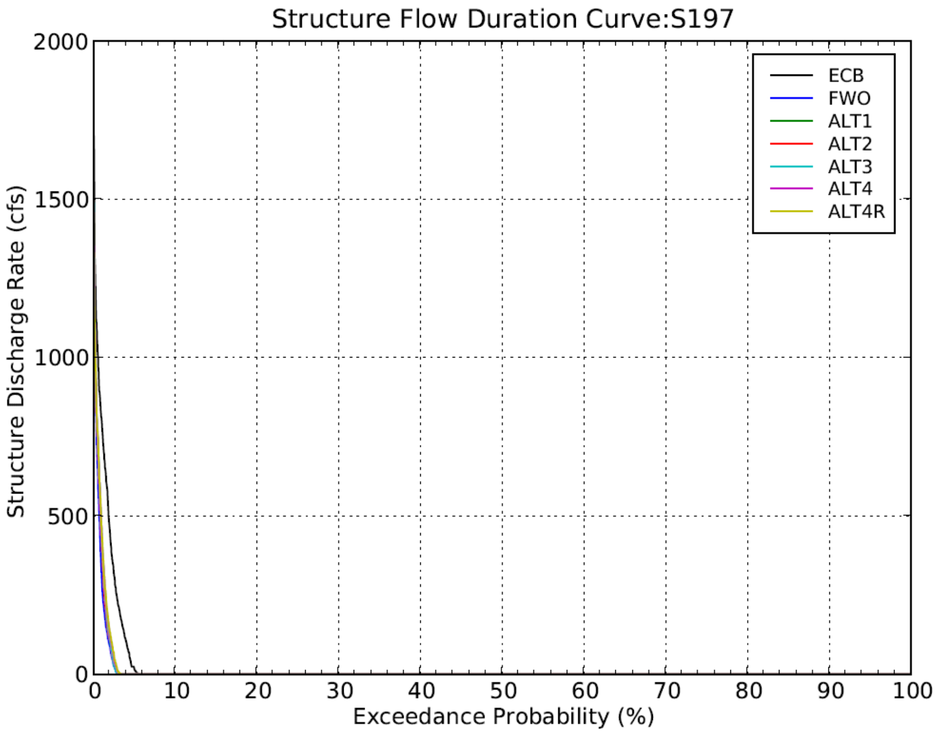


Figure 130A: Flow duration curve for SDCS C-111 Canal outlet structure S-197 to Florida Bay

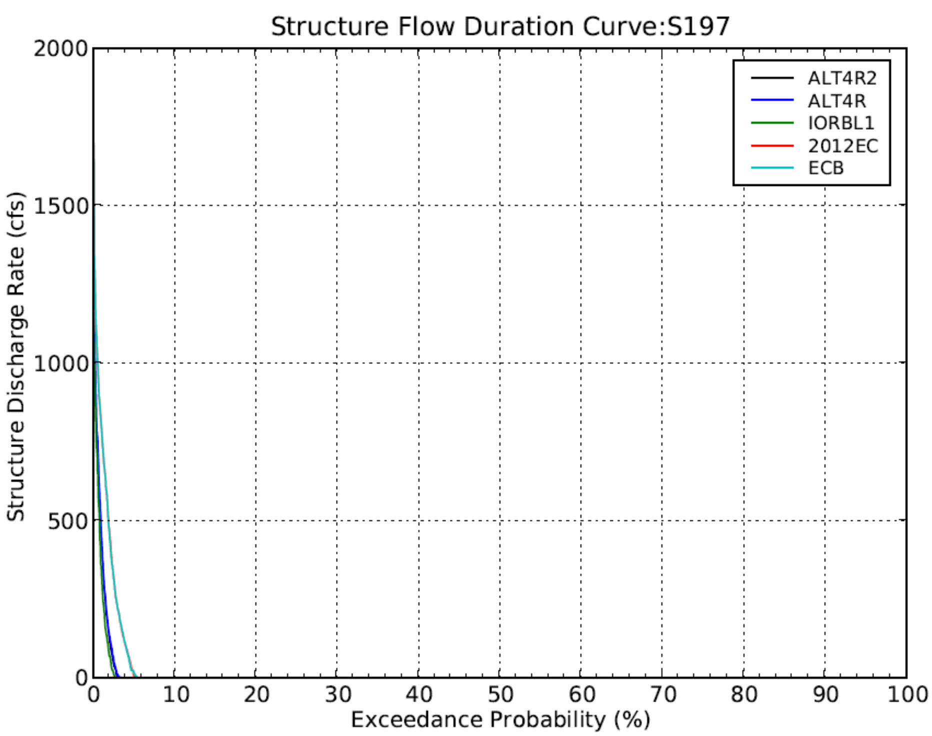


Figure 130B: Flow duration curve for SDCS C-111 Canal outlet structure S-197 to Florida Bay

ANNEX A-2: REFERENCE 1

CEPP MODELING STRATEGY

Central Everglades Planning Project Modeling Strategy

10/26/2012

Central Everglades Planning Project
Modeling Strategy
10/26/2012

Table of Contents

1.0 Introduction

The Central Everglades Planning Project (CEPP) is an expedited planning effort undertaken as part of the overall Comprehensive Everglades Restoration Plan (CERP), a program led by the United States Army Corps of Engineers (USACE) with the South Florida Water Management District (SFWMD) as local sponsor. This effort will seek to develop a Project Implementation Report (PIR) that combines planning and design activities for three primary areas of interest as follows: 1) Storage & water treatment facilities in the Everglades Agricultural Area, 2) Decompartmentalization of levees within the Everglades Protection Area (EPA) and 3) Levee seepage management features along the Everglades / urban boundary in southeastern Florida. Modeling support to the CEPP effort will be primarily provided by a team comprised of modelers from the Hydrologic & Environmental Systems Modeling Section of the SFWMD and the Interagency Modeling Center (IMC), although direct support from USACE, Department of the Interior (DOI) or contracted staff will likely be needed as the project requirements are more fully identified.

Due to the expedited nature of the CEPP, all modeling activities will need to be completed within an aggressive twelve month schedule (**Figure 1.1**). Additionally, it is anticipated that the range of alternatives to be assessed may be greater than in more traditional CERP planning efforts due to the project goal of increased public interaction in the planning process for CEPP. In consideration of these factors, modeling tools that provide flexible inputs to accommodate uncertain planning outcomes were selected over other available tools. It is also recognized that the evaluation strategies for the CEPP are still being developed, with the level of complexity and scope of evaluation remaining undefined as of the development of this strategy. Despite this, it is expected that the modeling tools described in this document will provide adequate hydrologic information to feed evaluation of the entire south Florida system for the needs of the CEPP.

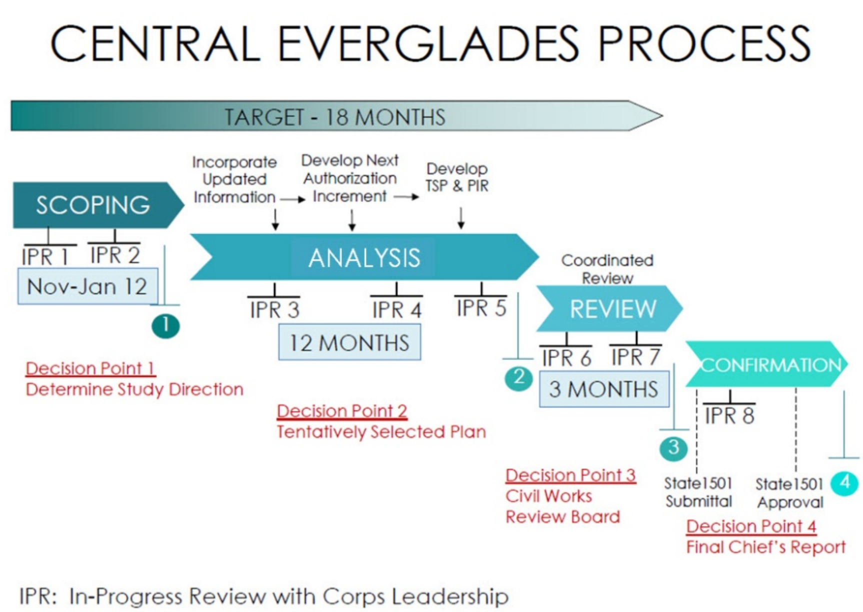


Figure 1.1 Completion of modeling activities within an aggressive 12 month schedule.

2.0 Use of Models in CEPP

The primary application of models in the CEPP will be in the assessment of regional-level hydrologic planning. More detailed models will also be brought to bear on specific questions related to hydraulic and water quality constraints. At this time, the modeling strategy does not consider the application of detailed flood event modeling (or hydrodynamic levee assessment) or water quality fate / succession modeling within the EPA given the schedule of the CEPP. Depending on the outcomes of the CEPP scoping phase and risk registry development, it is possible that key elements of this strategy may need to be revisited.

In general, the primary elements of the CEPP modeling support fall under the following three categories of the analysis phase of CEPP: 1) Updated Conceptual CERP Framework, 2) Plan Formulation (for next construction increment) and 3) Project Assurances for Tentatively Selected Plan (TSP). The specific model applications associated with each of these categories are listed in **Table 2.1** (model descriptions will be provided later in this document). In order to account for the entire south Florida domain from a planning modeling perspective, a decoupled approach as shown in **Figure 2.1** is proposed utilizing one set of tools to model the northern portion of the system with a different set of tools to represent the southern portion of the system. These tools will communicate iteratively using a set of shared boundary conditions along the EPA border – a transect known in the south Florida stakeholder community as the "Red Line". A methodology for resolving the temporal and spatial characteristics of flow at the red line in order to provide seamless translation of boundary conditions across models will need to be developed as part of the modeling effort.

From a schedule perspective, the CEPP has a very aggressive modeling timeframe (**Table 2.2**). Obviously, this will limit the number of alternatives that can be reasonably assessed and the level at which the evaluation can occur. Due to this consideration, CEPP will utilize a modified approach to the traditional modeling workflow of narrowing and refining alternatives incrementally over longer periods of time. In CEPP, where possible, batch processing of model information and/or inverse modeling techniques will be performed up front to identify to decision-makers key performance or tradeoff issues. An example of this type of approach can be observed in **Figure 2.2** where the orange line was developed by running hundreds of model scenarios to identify the trend in expected performance for a given performance metric with the inclusion of simulated storage. Once these types of curves have been developed, alternative development can be facilitated by selecting desired points on the curve based on project objectives or cost constraints. As such, a typical modeling "cycle" for CEPP would involve months of up-front work to develop these types of curves and model alternatives close to those anticipated in the plan. Then, alternative-specific modeling can be completed in a shorter turnaround (4 to 6 weeks for 3-4 alternatives in a given process step) since close-to-alternative modeling already exists. In this paradigm, QAQC of modeling outcomes will not be sacrificed, but full documentation may be deferred to occur later in the 12 month modeling schedule and not coincident with the development of each alternative.

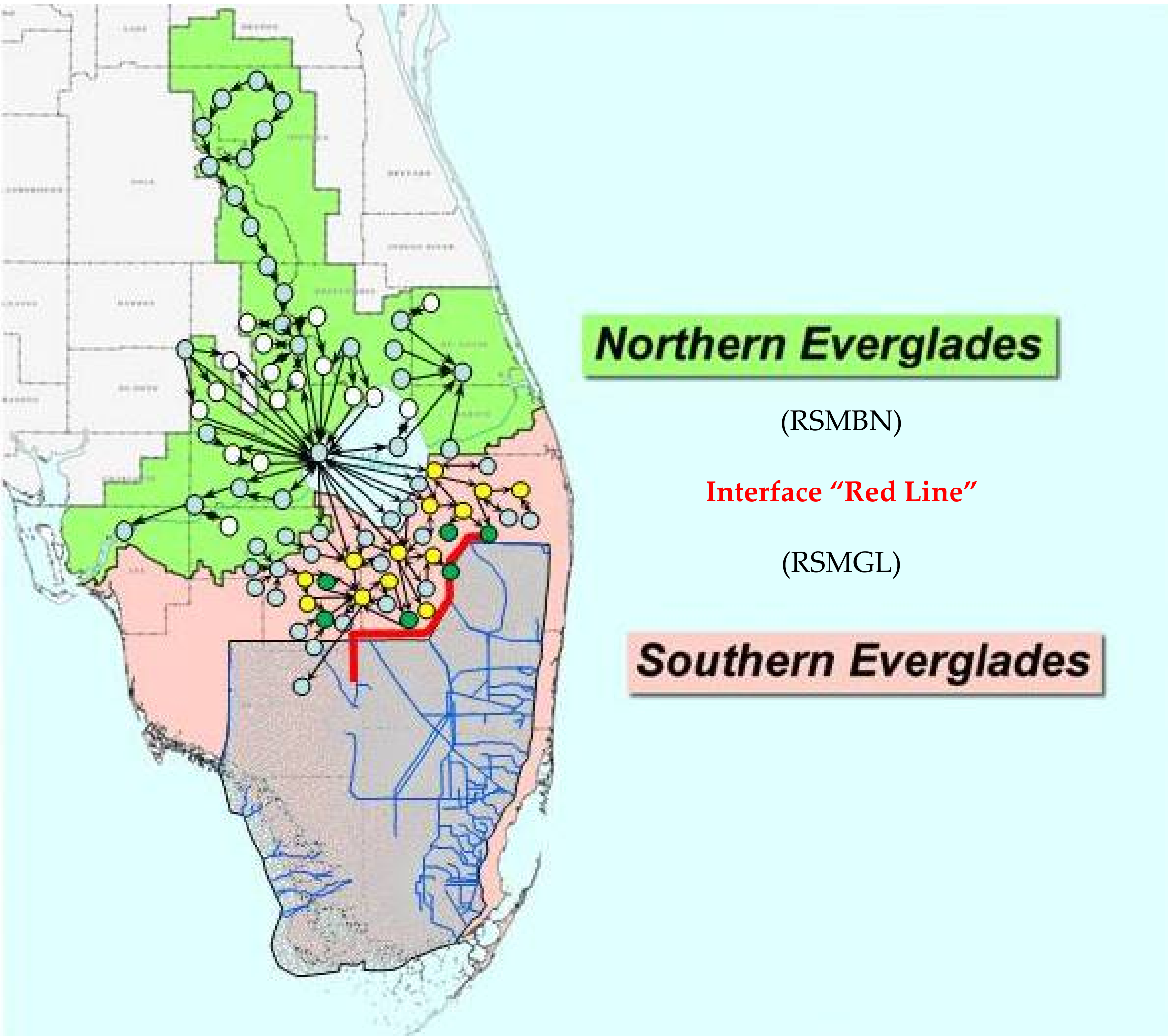


Figure 2.1. Decoupled CEPP Modeling Approach

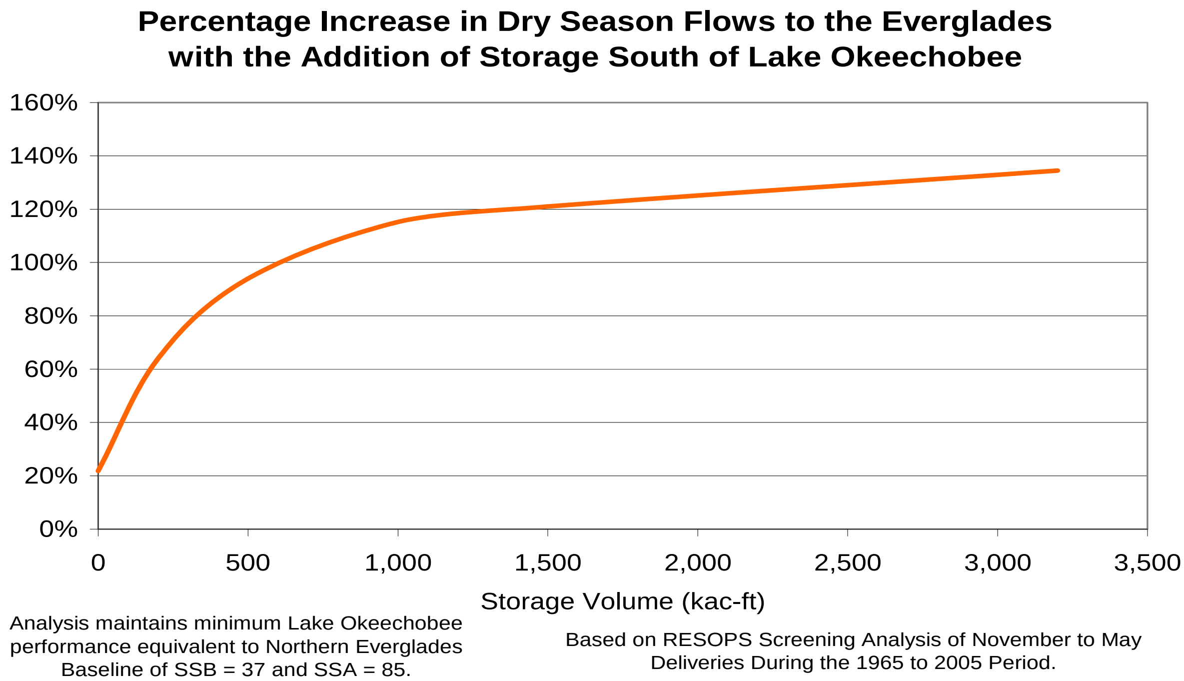


Figure 2.2. Example of Batch-Processing Model Application to Inform Decision Making

Table 2.1. Anticipated Modeling during the analysis phase of the Central Everglades Planning Project.

	Goals	Strategy	Model
Project Analysis Phase	**Updated Conceptual Framework** (~ 3 Months) • Restoration Flow Targets • Everglades Flow Scenarios	To provide modeling representations of the range of long-term restoration goals (of which CEPP will achieve an increment), the SFWMM will be used to represent the CERP configuration and the RSMGL will be used to represent updated concepts (e.g. River of Grass scenarios). RESOPS could be used to provide information of long-term northern storage and treatment needs.	RSMGL SFWMM RESOPS
	Plan Formulation (~6 Months) (Develop Next Increment) • EAA Storage and Treatment o Identify Formulation Scope/Constraints o Alternatives Screening o Alternatives Formulation/Evaluation o Identify Preferred Concept • DECOMP & Seepage Management o Identify Formulation Scope/Constraints o Alternatives Screening o Alternatives Formulation/Evaluation o Identify Preferred Concept	In plan formulation of the CEPP increment, up-front screening of alternatives above the red line will be performed primarily using the RESOPS, LOOPS and C-43 models. Additionally, use of batch processing and inverse modeling techniques will allow DMSTA to be applied during the screening phase of the effort to answer water quality questions north of the red line. Similar techniques will be applied to iModel and RSMGL to provide screening input south of the red line. Flow volumes will be translated across the models as boundary conditions and iteration between solutions north and south of the red line may be needed. Upon completion of the screening phase and identification of input assumptions for alternative assessment, final alternatives will be modeled using the RSMBN and RSMGL with detailed evaluation information being post-processed. Simulation of these alternatives will incorporate information gained from the screening, and DMSTA applications. HEC-RAS may also need to be applied in this final step to inform conveyance limitations or design requirements to the representation of alternatives.	RESOPS LOOPS C-43 RSMBN DMSTA HEC-RAS iModel RSMGL
	Project Assurances (~3 Months) o Finalize environmental assessments o Project Assurances o Water Made Available o Interim Operating Plan	Assurances assessment for saving clause, water made available and flood protection will primarily rely on post processing of the RSMBN and RSMGL representation of the CEPP Tentatively Selected Plan. Depending on public interest and management direction, other detailed models may also be needed for assessment of flood protection.	RSMBN RSMGL

Table 2.2. Original key modeling milestones and deliverables of the analysis phase of the CEPP SFWMD/IMC Modeling Team. This schedule was revised in August due to a change in project resourcing (**Table 2.3**).

ID	Task Name
1	FINISH Existing Conditions Baseline (ECB) assumptions table
2	FINISH Future without Project Baseline (FWO) assumptions table
3	Review and refine ECB
4	Review and refine FWO
5	FINISH alternatives identified above red-line
6	Review and refine alts above red-line
7	FINISH alternatives identified below red-line
8	FINISH identify Tentatively Selected Plan (TSP)
9	Review and refine combined alts
10	FINISH review and refine RSMGL screening below red-line
11	Review and refine final combined alts
12	Review and refine DMSTA screening
13	Review and refine RESOPS screening
14	Review and refine combined TSP assurances modeling
15	Review and refine docs for RSMGL, RSMBN, RSOPS, DMSTA, HEC-RAS
16	Review and refine final combined TSP assurances
17	FINISH Modeling Strategy
18	START integrate updated science into models
19	FINISH post plan formulation modeling results to DASR
20	FINISH post TSP modeling results to DASR
21	FINISH archive all model documentation to DASR / Documentum
22	RSMGL ECB Modeling
23	DRAFT RSMGL ECB ready for team review
24	RSMGL FWO Modeling
25	DRAFT RSMGL FWO ready for team review
26	Screening below red-line
27	RSMGL model documentation
28	RSMGL assurances model documentation
29	DRAFT RSMGL assurances model documentation
30	RSMBN ECB Modeling
31	DRAFT RSMBN ECB ready for team review
32	RSMBN FWO Modeling
33	DRAFT RSMBN FWO ready for team review
34	RSMBN alternative modeling
35	DRAFT RSMBN alternatives above red-line ready for team review
36	RSMBN model documentation
37	RSMBN assurances model documentation
38	DRAFT RSMBN assurances model documentation
39	RSMBN & RSMGL combined alternative modeling
40	DRAFT RSMBN & RSMGL combined alternative modeling ready for team review
41	QAQC RSMBN & RSMGL combined alternative modeling
42	RSMBN & RSMGL combined TSP assurances modeling
43	DRAFT RSMBN & RSMGL combined TSP assurances modeling ready for team review
44	DMSTA screening above red-line
45	DMSTA formal application for screening above red-line
46	DMSTA model documentation
47	DMSTA check RSMBN & RSMGL WQ results
48	RESOPS screening above red-line
49	RESOPS formal application for screening above red-line
50	RESOPS model documentation
51	Testing of PMs and Evaluation Methods
52	hydraulic conveyance testing, if needed
53	HEC-RAS model documentation

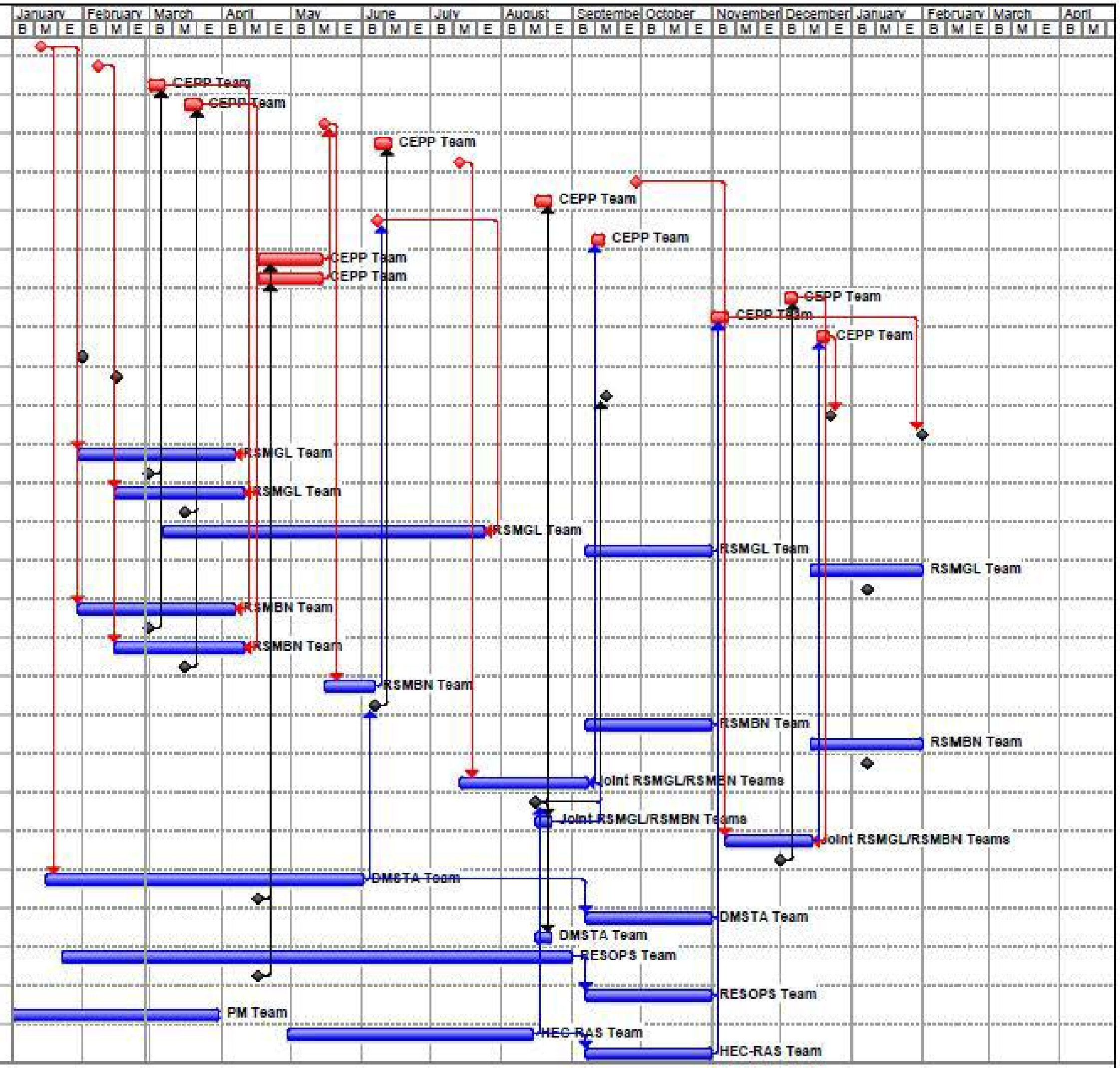


Table 2.3. Revised key modeling milestones and deliverables of the analysis phase of the CEPP SFWMD/IMC Modeling Team.

CEPP Staffing Level	
SFWMD	4 FTE
USACE IMC & DOI	3 FTE
Commitment of Resources	Through March 2013

Task Group	CEPP Modeling Work Product	Deliverable	Date
1.0	**Baselines**		
	1.1	FWO Update with A1 FEB	10/12/12
	1.2	Updated ECB and FWO (per team feedback)	11/15/12
2.0	**North Redline Screening / Alternatives**		
	2.1	Meet with Ag	8/3/12
	2.2	Meet with Est	8/9/12
	2.3	FEB scenario with RSMBN	8/22/12
	2.4	FEB scenario with RSMBN inc Holeyland	8/28/12
	2.5	Present to PDT	8/28/12
3.0	**Blueline Screening**		
	3.1	NSM0 - NSM100 Sensitivities	7/27/12
	3.2	Additional Sensitivities as needed	9/30/12
4.0	**South Redline Screening**		
	4.1	RL1 - RL4 Sensitivities	7/31/12
5.0	**L28 Sensitivities**		
	5.1	Updated to RSMGL	8/15/12
	5.2	L28 Scenarios	8/22/12
	5.3	Meet with Tribe	8/23/12
	5.4	Present to PDT	8/28/12
6.0	**Greenline Screening**		
	6.1	Meet with CEPP Team to set targets & conceptual configs	8/15/12
	6.2	WG presentation on targets / ideas	8/30/12
	6.3	Complete iModel implementation	9/6/12
	6.4	iModel application to CEPP team	9/19/12
	6.5	Present to PDT	10/2/12
	6.6	Meet with CEPP Team to set Measures	10/10/12
	6.7	Present Measures to PDT	10/25/12
7.0	**Yellow Line Sensitivities**		
	7.1	Alternative Testing	11/15/12
8.0	Final Array of Alternatives		Mid-December
9.0	Project Assurances		Mid-February
10.0	Technical Review / Model Certification		End-December
11.0	Documentation		End March

3.0 Screening Model Overview

The screening techniques outlined in section 2.0 can be applied to any of the planning models identified in this strategy document. In addition to use of these methodologies, the specific use of the RESOPS and LOOPS models for screening purposes are proposed.

3.1 REservoir Sizing and OPerations Screening (RESOPS)

To assist with the preliminary analyses and testing of alternative storage configurations that consider the interconnectivity of Lake Okeechobee, the Lake Okeechobee Service Area, the northern estuary watershed systems, and the Everglades, a spreadsheet-based screening model was developed. The REservoir Sizing and OPerations Screening (RESOPS) Model is a coarse-scale water management simulation model that was developed to quickly test alternative reservoir sizes and system operating rules for the region surrounding and including Lake Okeechobee (**Figure 3.1.1**).

The RESOPS Model is, as its name states, a screening-level model. The RESOPS Model has a limited scope and is not a replacement for the detailed regional hydrologic simulation models that have traditionally been used for the analysis and planning of south Florida's water resources. Those detailed models, the South Florida Water Management Model (SFWMM or 2x2) and the Regional Simulation Model (RSM), are necessary for the comprehensive in-depth analysis of the existing and future components of the water management system in south Florida. Although the detailed regional models are the best available tools for performing the finer-scale evaluation, they are not as appropriate for quickly testing a broad range of alternative reservoir sizes and Lake Okeechobee operation configurations. The strength of the RESOPS Model is with its ability to quickly test the performance of alternative configurations which can provide guidance for more-detailed modeling. Such a hierarchical process can allow the detailed models to focus on a smaller number of more-promising alternative plans.

The RESOPS Model was built using Microsoft® Excel 2003. It performs monthly time-step, 41-year continuous simulations of the hydrology and operations of south Florida's regional water management system and the interaction with proposed reservoir and wetland treatment area features. Within one second, the RESOPS Model executes a simulation and automatically produces a wide variety of graphical and statistical summary measures of performance that can be used to compare up to four test scenarios. The model also contains an optimization routine that enables selection of superior operating rules for Lake Okeechobee. Performance summary graphics are another useful feature which facilitates the comparison of multiple simulations. Much of the basic input data to RESOPS is provided by the detailed regional simulation models, specifically the SFWMM. Although the RESOPS Model simulates flows to the northern boundary of the Everglades Water Conservation Areas (WCAs), it does not simulate the complexity of the spatial distribution of flows and stages within the Everglades.

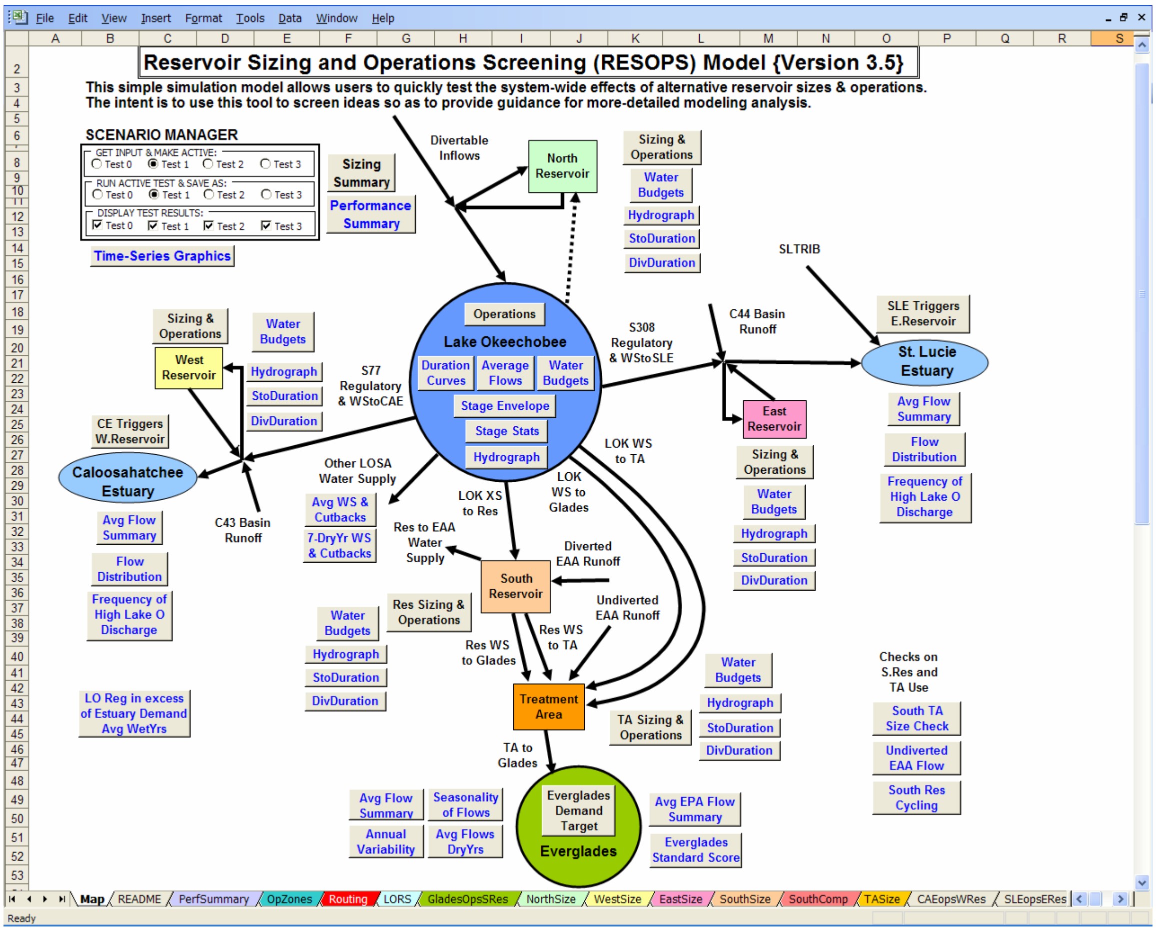


Figure 3.1.1. Schematic representation of study area hydrology as seen by the RESOPS Model.

3.2 Lake Okeechobee OPerations Screening (LOOPS)

The LOOPS model **(Neidrauer et al, 2006)** is a hydrologic simulation tool that provides rapid screening-level testing of alternative operating rules and strategies for Lake Okeechobee, including Regulation Schedules, Water Shortage Plans, and protocols for defining release amounts when the Regulation Schedule guidance only provides ranges of flows. Inputs include daily time-series values for the Lake net inflow, basin runoff from the Caloosahatchee and St. Lucie basins, lake evaporation rates, and the hydrologic state and forecast information that drive Lake regulation schedules. The strength of the LOOPS Model is with its ability to quickly test the performance of alternative operating scenarios to screen ideas and perform sensitivity tests for the primary lake-management objectives.

The LOOPS Model was developed with Excel 2003, and performs 41-year continuous simulations with a daily time-step of the hydrology and operations of the water management system, including Lake Okeechobee, the Lake Okeechobee Service Area, and the Caloosahatchee and St. Lucie watersheds and estuaries. The time-series of Lake releases south, to the WCAs via STA-3/4 and to C-51 via L-8, are assumed boundary conditions and are derived from the SFWMM or the RESOPS model as appropriate for the application. An input parameter/multiplier is available to adjust these time-series.

The basic structure of LOOPS is illustrated in **Figure 3.2.1**. Data management is simple and transparent to the user. Macros do the work of copying the pertinent information from the "active schedule" sheet to separate sheets for each alternative.

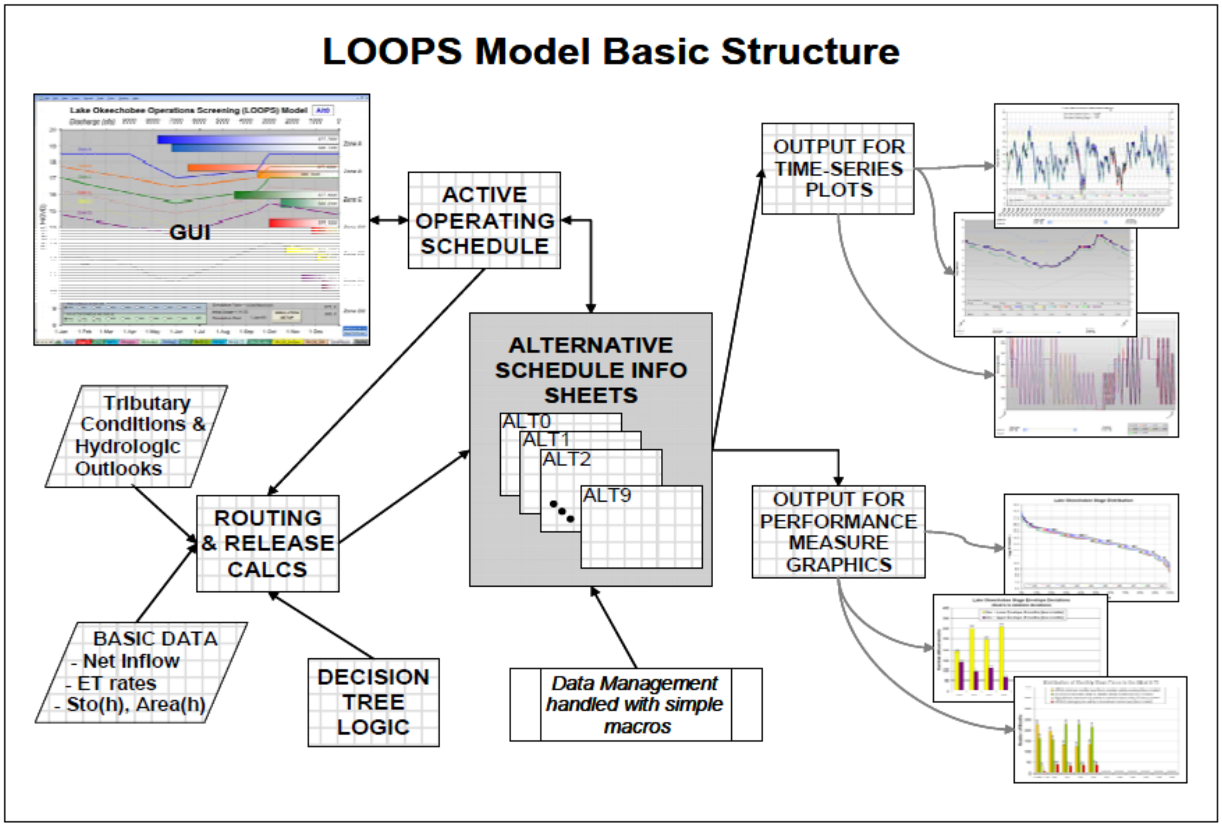


Figure 3.2.1. Basic Structure of the LOOPS Model

The LOOPS model can be run in batch mode by setting ranges for various parameters to be varied. Key outputs are identified and values are stored for each run processed in batch. At the end of the set of runs, all of the values can be plotted and compared to other key outputs to find optimal parameter values for all key modeling objectives. A small subset of optimal parameter values can then be run using more complex models to make policy decisions. **Figure 3.2.2** shows an example graphic comparing tradeoffs between two performance measures. LOOPS allows setting preferred ranges on the performance measures, and all points that fall within the ranges of all the performance measures show on the graphic in red.

Because the LOOPS model does not simulate storage in the C-43 basin, it was necessary to use the C-43 Spreadsheet Model for an accurate depiction of changes in the effects of Lake Okeechobee releases to the west. That model is described briefly in the next section. However, since C-44 project features are not designed to capture Lake Okeechobee releases, any discharges to the St. Lucie Estuary to the east will discharge directly at structure S-80, and therefore did not require the use of a specialized model.

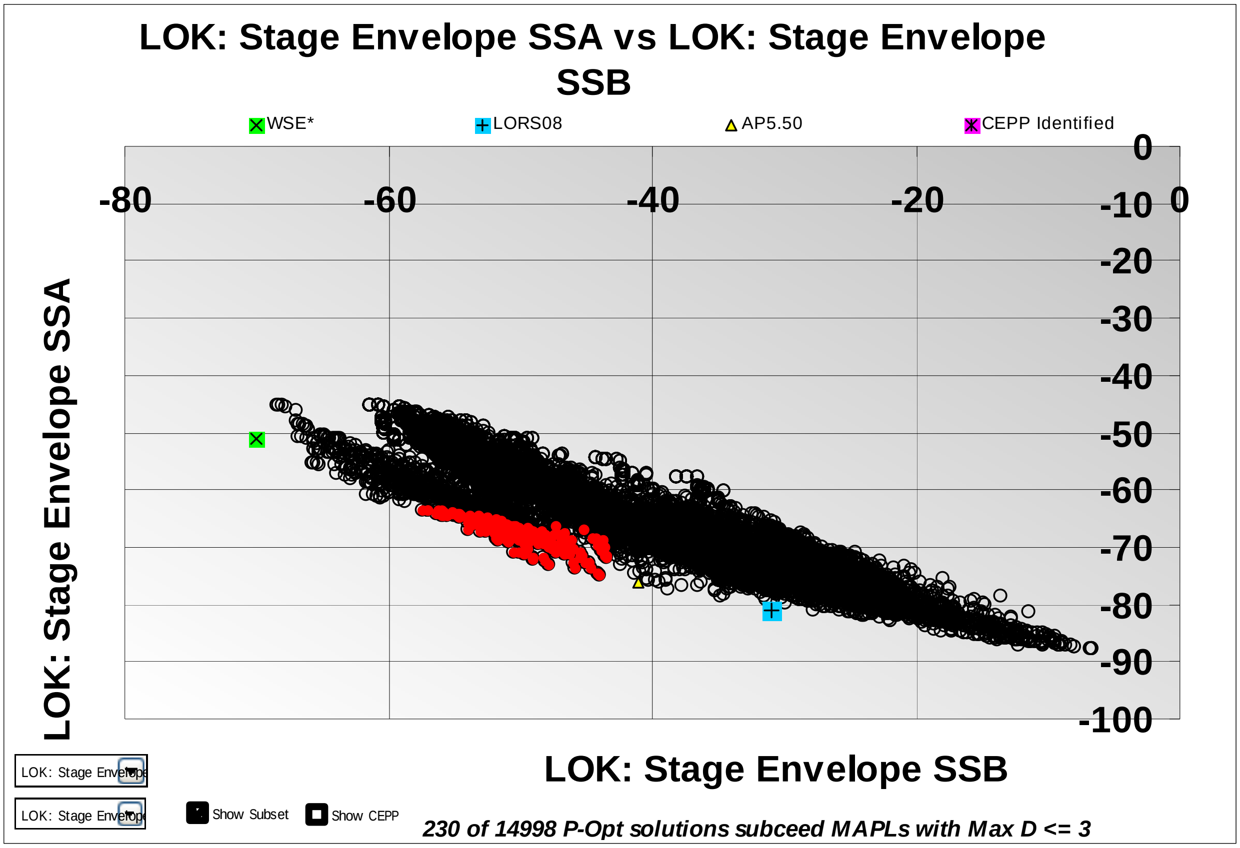


Figure 3.2.2. Example graphic showing tradeoffs between two performance measures after completion of batch runs of different parameter combinations.

3.3 C-43 Spreadsheet Model

The C-43 Spreadsheet Model “C43_PIR-model_Final.xls” was developed for the CERP Project “C-43 Reservoir Phase I” **(Starnes & Marlowe, 2007)** to compare with-project discharge over S-79 (the downstream point at which the basin discharges into the estuary) to both the pre-project discharge over S-79 and to a time series representing restoration target flows over S-79 for a 41-year, daily period of simulation. The model also shows a water budget for the reservoir and tracks reservoir inflows, releases and storage.

3.4 iModel©

Instead of modeling the cause effect-relationship typically identified in physically based engineering models such as the RSM, the iModel is a unique inverse modeling tool that reverses the process of a traditional, direct model. iModel computes a system's required input to achieve a system's desired response. iModel emulates the reversal of the physical process using Artificial Neural Networks and Genetic algorithm theories. The iModel is comprised of two primary components:

1) Hydrologic Model Emulators (HME) which are developed for each waterbody of interest (e.g., Everglades areas) to predict stage and transect flows at the identified target locations as a function of inflows and outflows and net rainfall, as shown conceptually in **Figure 3.4.1** (Ali, 2009)
2) A neural network optimization framework that manages and optimizes concurrent performance of the different waterbodies' HMEs to approach the desired targets subject to linear and nonlinear constraints and weights.

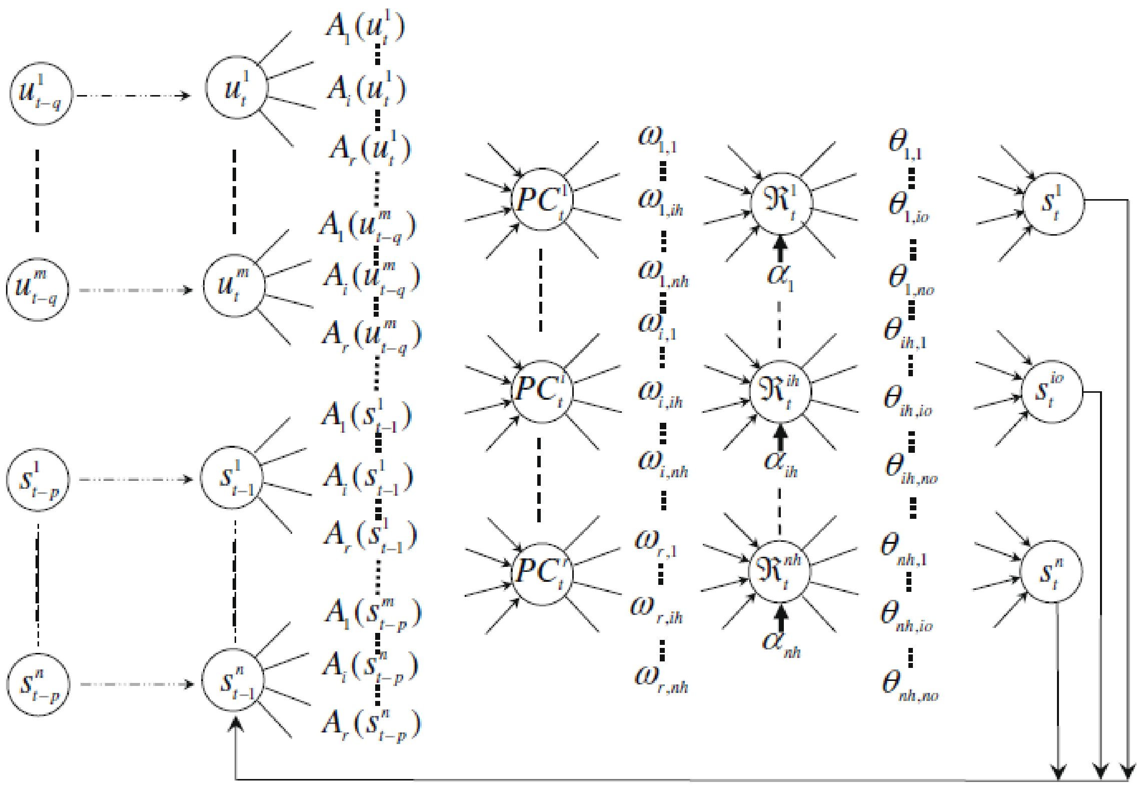


Figure 3.4.2 Schematic diagram for Hydrologic Model Emulator

The primary goal of iModel application is the efficient utilization of the available water and storage facilities to achieve desirable environmental benefits while maintaining flood control and water supply requirements. It can guide, and reduce the run time of, traditional models by offering optimal solutions that would otherwise require hundreds of traditional model runs. It provides simple operational functions for complex systems towards Operational Protocols Development. The iModel's predicted inflows/outflows can be used as input to a traditional direct model to check that the simulated response of the traditional model matches the desired response of the iModel.

Given the rapid pace of the CEPP process, traditional analysis of multiple configurations is a logistics challenge. The IModel will be utilized in CEPP screening to help identify a limited set of system operational protocols and optimal infrastructure that are needed to approach CEPP planning objectives. Ultimately these configurations will be modeled using the detailed physical model (RSMGL) prior to final project evaluation. By employing this approach, a wide range of potential outcomes can be quickly analyzed and a feasible range of high performing alternatives can be identified using a systematic and robust process.

4.0 Planning Models Overview

Regional hydrologic models are the primary modeling tools to be used for Central Everglades assessment. The models provide daily, detailed estimates of hydrology across the planning domain. They simulate detailed daily rainfall-runoff processes and flow routing within the Central Everglades planning region as a function of existing infrastructure and proposed configurations. The strategy is to use a decoupled link-node model (RSMBN) for the EAA, STAs and northern areas in combination with a detailed meshed model (RSMGL) for the Everglades-Lower East Coast areas.

4.1 RSM Basin (RSMBN)

The Regional Simulation Model -Basins (RSM Basins or RSMBN) uses the same source code as the mesh-based Regional Simulation Model which, in turn, was recently implemented in the evaluation of alternatives for the Water Conservation Area 3 Decompartmentalization Project (using RSMGL described in section 4.2). Both implementations are based on object-oriented concepts and principles. The object-oriented nature of the model not only describes the physical connectivity of the waterbodies but, likewise, describes the computational engine of RSMBN. This feature allows new objects to be added without the need to significantly alter the previously coded modules and objects in the computer program. For example, adding the operation of a new reservoir would be simulated as adding a discrete “object” that is automatically assigned with the features and functions commonly defined for a reservoir in the water management system.

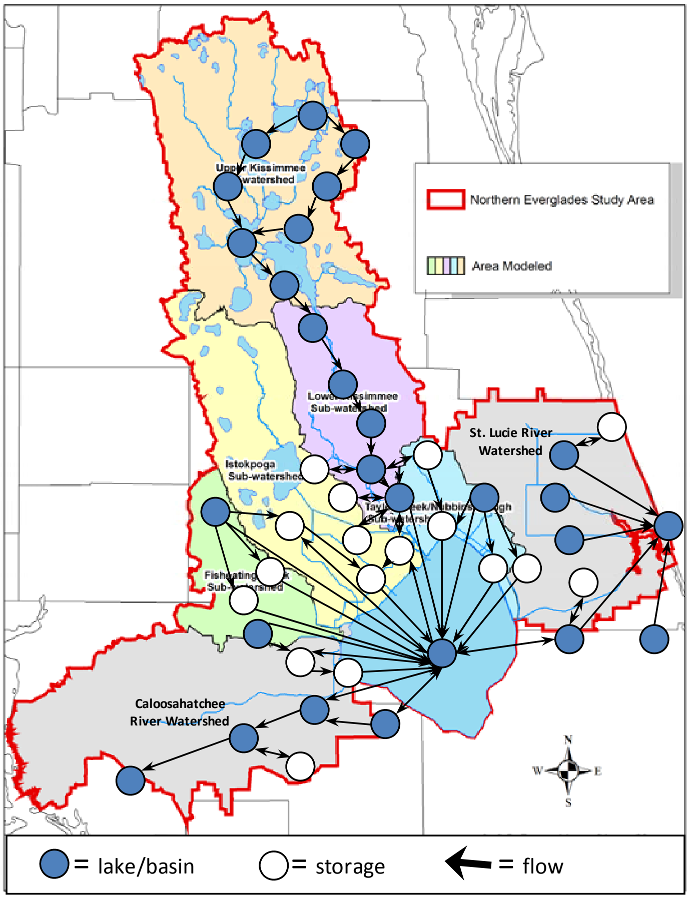
The RSMBN is a link-node based model designed to simulate the transfer of water from a pre-defined set of watersheds, lakes, reservoirs or any ”waterbody” that either receives or transmits water to another adjacent waterbody. The model assumes that water in each waterbody is held in level pools. The model domain covers Lake Okeechobee and four major watersheds: Kissimmee, Lake Okeechobee, St. Lucie River, Caloosahatchee River and the Everglades Agricultural Area, the latter being the latest addition. The watersheds are further divided into sub-watersheds until fundamental waterbodies can be considered as separate model nodes. Individual operating rules were encapsulated into the model that defines how water is moved between two nodes. Taken together, the set of management rules define the linkage of all nodes within the model domain.

The model is considered a lumped model in hydrologic engineering terms. Thus, local-scale features within a watershed, e.g. stages at individual gauging stations or flows across specific transects, are not simulated. Simulated stages represent average water level conditions for the entire waterbody. No systematic detailed verification relative to historical data was done during initial model set-up; however, the model was validated by making comparative runs with established legacy models currently in use within the model domain: the UKISS for the Upper Kissimmee Watershed (Fan, 1986) and selected sub-areas in South Florida Water Management Model (SFWMD, 2005). Additionally, historical information (in some cases, full calibration efforts) has been used in the development of nodes representing the C-139 basin, Stormwater Treatment Areas and 298 districts within the EAA, a procedure never employed in previous regional hydrologic modeling of these areas.

Input data for the model includes daily records of hydrologic and meteorological data (rainfall and potential evapotranspiration), as well as discharges at the boundaries for a 41-year period between 1965 and 2005. Other model input data includes the physical description of management features (e.g., reservoir stage-storage relationship and structure capacities) and corresponding operating rules (e.g., maximum operating levels and reservoir outflow priorities).

Runoff and supplemental irrigation demands can be simulated in the different waterbodies in RSMBN, or they can be read-in as time series boundary conditions, as in the case for the Caloosahatchee and St. Lucie basins (where hydrology is calibrated offline using the AFSIRS-WATBAL model (Smajstrla, 1990)). Stages in waterbodies and flows at inlet and outlet structures are basic output data from the model.

The RSMBN model provides a very good tool for assessing the water budget interaction in a complex hydrologic system. The model input requirements are not as rigorous and computational needs are not as CPU-intensive as other mesh–based models. The model executes in only a few minutes for a system representation as in **Figures 4.1.1 and 4.1.2**. The model is also an effective tool in comparing the relative performance of the proposed alternatives. In order to make an effective comparison, raw model outputs are summarized in a way that fits the basins or metrics associated with the selected performance measure. Post-processing scripts are available that temporally (weekly, monthly, seasonal, etc.) and spatially (individual waterbody or collection of waterbodies) summarize model output. Generation of some performance measure graphics are automated as they have been previously defined and vetted in other model application projects, e.g. CERP, LECPLAN, etc. The RSMBN precursor, the Northern Everglades Regional Simulation Model (NERSM), has been implemented to assess the hydrologic impact of selected alternatives for SFWMD planning efforts under the Northern Everglades program, specifically the Lake Okeechobee Phase 2 Technical Plan (LOP2TP) and the River Watershed Protection Plan (RWPP).



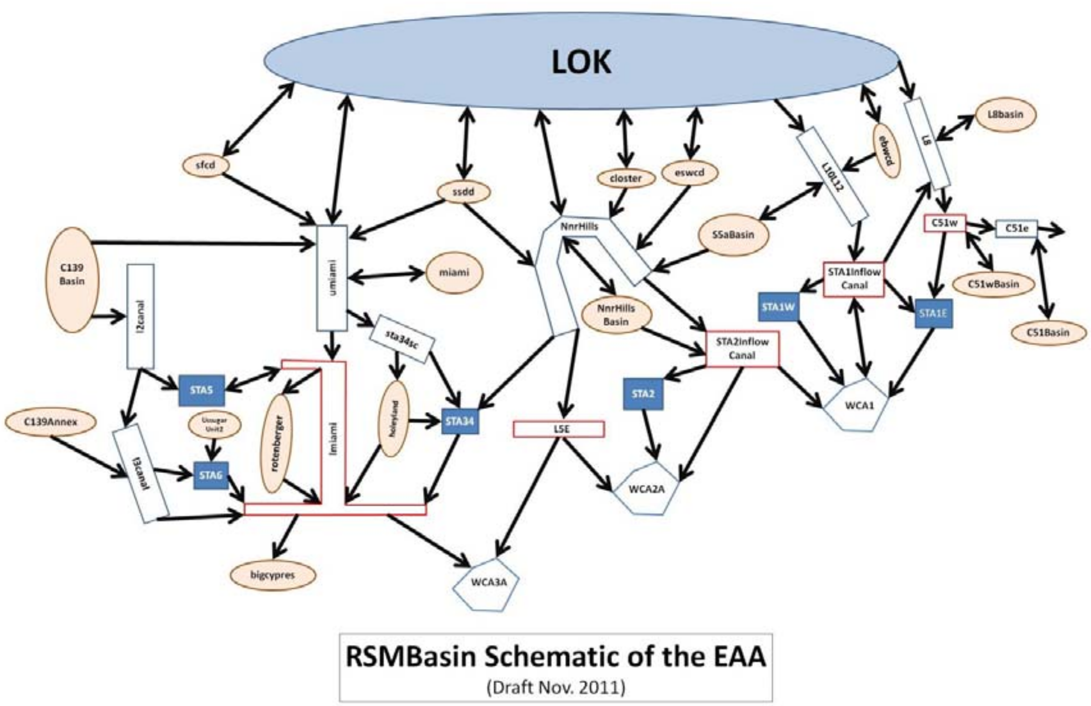


Figure 4.1.1. & Figure 4.1.2. RSM Link-Node Representation for Central Everglades Project

4.2 RSM Glades-LECSA (RSMGL)

The Glades-LECSA model provides a tool to simulate the natural hydrology and the water management operations of several important basins in south Florida. The Glades-LECSA implementation uses the Regional Simulation Model (RSM) developed by the Hydrologic and Environmental Systems Modeling Section of the South Florida Water Management District. The RSM is an implicit, finite-volume, continuous, distributed, and integrated surface-water and ground-water model. It can simulate one-dimensional canal/stream flow and two-dimensional overland and groundwater flow in arbitrarily shaped areas using a variable triangular mesh. The overland and groundwater flow components are fully coupled in the RSM for a more realistic representation of runoff generation. It has physically-based formulations for the simulation of overland and groundwater flow, evapotranspiration, infiltration, levee seepage, and canal and structure flows. The model uses the diffusive wave approximation of Saint-Venant's equation to simulate canal and overland flows. This model is capable of simulating features that are unique to south Florida such as low-relief topography, high water tables, saturation-excess runoff, depth-dependent roughness and very permeable soils. The Glades-LECSA model also has the capability to predict the results of implementing physical and operational alternatives being considered to address changing water management priorities and issues in south Florida's regional system. The Glades-LECSA model has been applied in PIR1 of the WCA-3 Decompartmentalization and Sheetflow Enhancement (Decomp) Project.

The Glades-LECSA model encompasses an area of 5,825 square miles. It covers six counties (some partially) and 13 hydrologic basins. The area of the model-domain includes the Everglades National Park, Water Conservation Areas, Big Cypress National Preserve, and the Lower East Coast Service Areas south of the C-51 canal in Palm Beach County (see **Figure 4.2.1**). The Glades-LECSA model mesh consists of 5,794 triangular cells with an average cell size of approximately one square mile. The mesh is designed to conform to all important flow controlling features, such as roads and levees within the model domain. The Glades-LECSA model simulates an extensive canal network. This network includes all primary canals that are maintained by the SFWMD. It also includes several secondary canals that are of importance. In addition, the model uses the Water Control District feature available in the RSM to simulate some secondary and tertiary canals as well. A one-day time step was used for the calibration and validation of the Glades-LECSA model. Other models (e.g., SFWMM) have used similar time steps in the past. Northern boundary flows are imposed based on output from the SFWMM or other regional models such as the RSM Basins model which incorporates areas north of the Water Conservation Areas.

Only the surficial aquifer is simulated in the Glades-LECSA model. Thus, the horizontal saturated hydraulic conductivity values as well as aquifer bottom elevation values that are used in this model are pertinent to that layer only. The Water Conservation Areas as well as the Everglades National Park contain a significant peat layer that affects stages within those areas. Consequently, it is simulated explicitly in the Glades-LECSA model using a stage-volume converter feature that is unique to the RSM. The model-domain contains several hundred Public Water Supply (PWS) wells that tap the surficial aquifer. These are also simulated in the model through the use of time-series data. The model-domain contains several roads and levees that act as overland flow barriers. The canal and regional groundwater seepage contributions across these levees are explicitly simulated in the Glades-LECSA model.

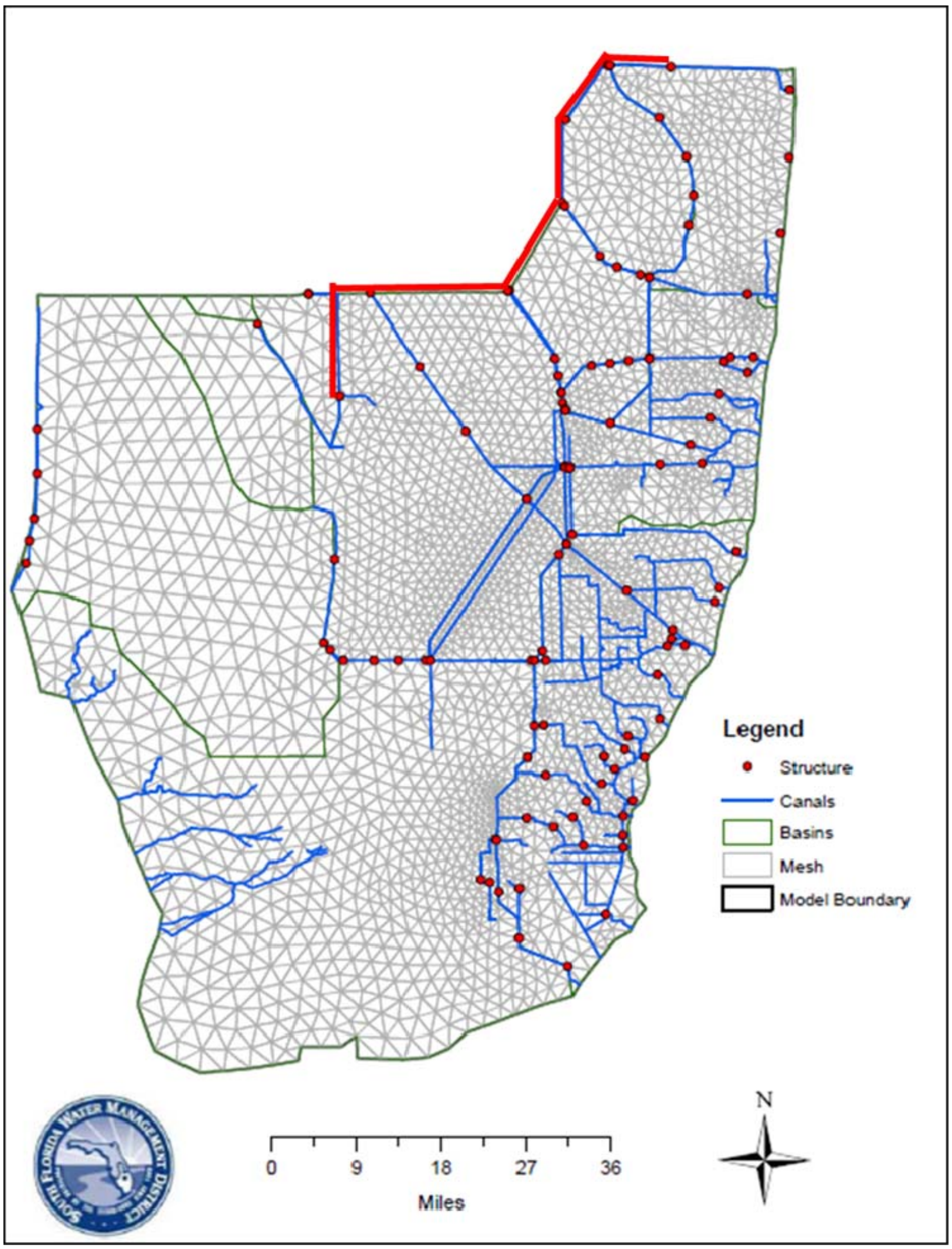


Figure 4.2.1. Glades-LECSA Modeling Domain with Major Canals and Structures

4.3 South Florida Water Management Model (SFWMM)

For the Central Everglades project, the South Florida Water Management Model (SFWMM) will be used as a source of boundary conditions to the other planning or detailed models and also as the representation of the full CERP condition in the "updating conceptual framework" portion of the project. The SFWMM is a physically-based simulation model that combines the hydrology and management aspects of a greater portion of the South Florida Water Management District (SFWMD). The model is regional in spatial extent (covering most of south Florida) and it encompasses an area of substantial heterogeneity in both natural and managed hydrology. It covers an area of 7600 square miles using a mesh of 2 mile x 2 mile cells. **Figure 4.3.1** shows the model boundary relative to south Florida.

The SFWMM is a coupled surface water-groundwater model which incorporates overland flow, canal routing, unsaturated zone accounting and two-dimensional single layer aquifer flow. The model simulates the major components of the hydrologic cycle in south Florida including rainfall, evapotranspiration, infiltration, overland and groundwater flow, canal flow, canal groundwater seepage, levee seepage and groundwater pumping. The model is site-specific because it was exclusively developed for the south Florida region. In addition to simulating the natural hydrology in south Florida, the model also simulates the management processes that satisfy policy-based rules (both existing and proposed) to meet flood control, water supply and environmental needs. It incorporates current or proposed water management control structures and current or proposed operational rules.

The model is conceptualized at varying levels of detail for three different geographic areas, as shown in **Figure 4.3.1**:

(1) for Lake Okeechobee,
(2) for the combined extent of the Everglades Agricultural Area (EAA), the Everglades Protection Area (EPA) and the Lower East Coast (LEC) south of Lake Okeechobee, and
(3) for non-EAA Lake Okeechobee Service Area (LOSA) basins.

In addition, the model includes inflows from Kissimmee River, and runoff and demands in the Caloosahatchee River and St. Lucie canal basins. Lake Okeechobee is modeled as a lumped system, or regarded as a single point in space without dimensions where simulated water levels and/or flow rates are spatially averaged. The gridded portion of the model domain describes the extent of the finite difference solution to the governing overland and groundwater flow equations and is defined south of Lake Okeechobee. For the rest of LOSA excluding the EAA, a simple flow balance procedure is used. In these basins, pre-processed user-input demand and runoff characteristics are combined with appropriate system operational rules to calculate flow distributions.

The SFWMM simulates hydrology on a daily basis using climatic data for the 1965-2005 period which includes many droughts and wet periods. The model has been calibrated and verified using water level and discharge measurements at hundreds of locations distributed throughout the region within the model boundaries.

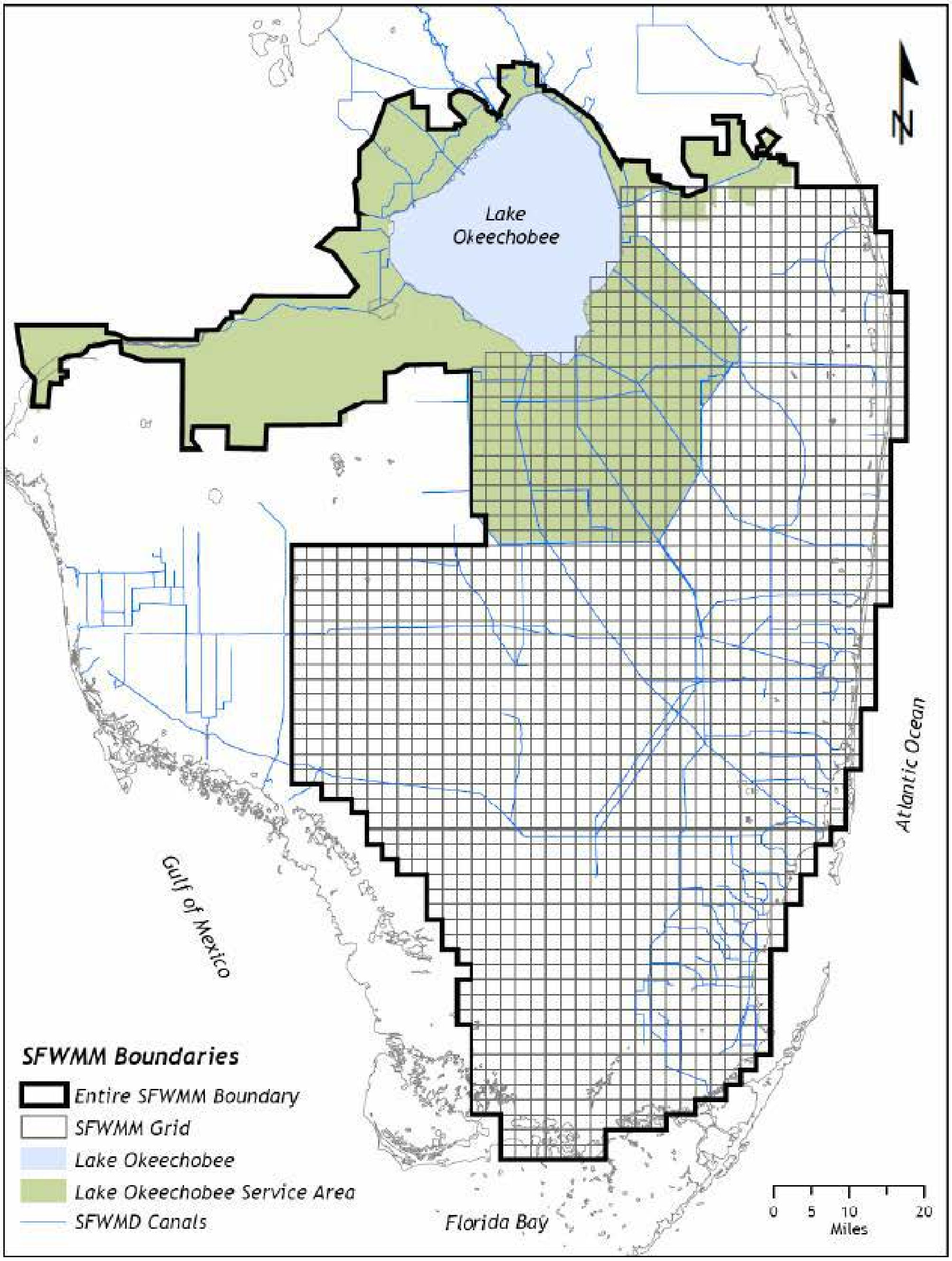


Figure 4.3.1. South Florida Water Management Model domain showing three geographic areas—1) Lake Okeechobee, 2) gridded portion of EAA, EPA, and LEC and 3) non-EAA LOSA basins.

5.0 Detailed Models Overview

On an as-needed basis, additional models may be applied to complement or assist the regional hydrologic models in analyzing system features. Examples of this type of model application will be shown for assessing water quality considerations and conveyance of water. The list of detailed models may expand or contract based on project requirements. The expedited schedule may also require that hydrologic surrogates be used in place of detailed modeling. Detailed flood assessment modeling is not envisioned within the Central Everglades Planning Project at this time.

5.1 Dynamic Model for Stormwater Treatment Areas (DMSTA)

The Dynamic Model for Stormwater Treatment Areas (DMSTA) was developed for the U.S. Department of the Interior and the U.S. Army Corps of Engineers (Walker and Kadlec 2005, http://wwwalker.net/dmsta/). DMSTA was developed and calibrated to information specific to south Florida and to predict phosphorus removal performance of Stormwater Treatment Areas (STAs) and storage reservoirs, and has been commonly used by both state and federal agencies for STA design and evaluation since 2001. The 2005 version of DMSTA was calibrated to data from 35 fully functional treatment cells with viable vegetation communities of various types. The model provides detailed output on the water and phosphorus balances of individual treatment cells and entire STAs, regional networks of STAs and storage reservoirs. Warning messages are generated in cases where simulated conditions exceed the calibration boundaries for phosphorus concentration, depth, dryout frequency, and/or flow velocities.

Model input requirements include daily values for flow, phosphorus concentration, rainfall, evapotranspiration (ET), depth (optional input or simulated value) and releases (optional input or simulated), treatment area configuration, cell size, flow path width, vegetation type, estimates of hydraulic mixing, outflow hydraulics, and seepage estimates. Phosphorus removal rates (settling rate; *K*) and other P cycling parameters can be either user-defined or calculated within DMSTA based on calibration data sets. DMSTA assumes that the specified vegetation types (emergent, submersed, periphyton) will be maintained in the long-term, but does not take into account areas subject to periodic disturbance such as hurricanes, droughts and other extreme conditions that are not reflected in the calibration datasets where vegetation management may be difficult.

DMSTA is the best available tool for simulating phosphorus removal performance of existing or planned storage basins and STAs. An input template has been developed to facilitate linkage to daily output from the regional hydrologic models. For the Central Everglades Planning Project, evaluation of planning-level water quality constraints in the Everglades Agricultural Area will be accomplished through the use of DMSTA. Ultimately, the amount of flow directed into the northern boundary of the Everglades Protection Area will be dependent on the assumptions made relative to water quality objectives and the resulting application of the DMSTA model.

5.2 Hydrologic Engineering Center's River Analysis System (HEC-RAS)

The Everglades Agricultural Area (EAA) is an area with complex drainage features comprised of mostly man-made canals and structures that move water from the primary canal system to the secondary and tertiary local drainage system via small canals and control structures. In many instances when draining storm water from large storm events, the drainage system may be overtaxed resulting in local flooding.

Evaluation of hydraulic capacity of canal systems can be performed with the Hydrologic Engineering Center's River Analysis System (HEC-RAS) model developed by USACE. The unsteady flow solver in HEC-RAS has been adapted from the UNET model which was developed to perform sub-critical or super-critical, gradually varied unsteady flow analysis. Hydraulic losses through the channel, bridge, culverts, spillways and other hydraulic structures can be modeled in both the steady state and unsteady state modules. In addition, the unsteady flow module can simulate storage areas, pumping stations, and levee failures. These model features can be useful in identifying conveyance deficiencies in canals and structures under steady state or dynamic flow conditions for particular flood events.

The most recent release of HEC-RAS (version 4.2) includes capabilities that allow the model to apply complex operation of gated structures and pump stations. Such operations can change in time or water level conditions anywhere in the system. A new feature in HEC-RAS will allow the 1-dimensional channel flow to interact with 2-dimensional floodplain flow, allowing for more accurate floodplain mapping. In areas where the interaction of open channel flow and aquifer groundwater needs to be explicitly modeled, a new integrated tool based on the original HEC-RAS and MODFLOW models can now be used to accurately simulate the aquifer/canal flow exchange. This new modeling tool is near completion and has been successfully used to simulate flood event conditions in the C-4 Basin in central Miami-Dade County. In this model application, flood control infrastructure and operations are being reevaluated to improve flooding conditions in municipalities along the C-4 canal. Application of RAS-MODFLOW may have to wait until HEC releases it into production.

For the Central Everglades project, evaluation of hydraulic performance of individual canal/structures can be accomplished with HEC-RAS under design flow conditions. Existing data from previous modeling efforts can be used to define the physical characteristics of canal and structures. Canal cross-sectional data and structure information are available from previous models such as Mike-11 and District's databases. HEC-RAS is applicable for 1-

dimensional flow conditions, however, for hydraulic analysis of flow-way conveyance that occurs in floodplains or STAs, the new 2-dimensional floodplain feature of HEC-RAS mentioned above can be employed.

5.3 MIKE Marsh Model of ENP (M3ENP)

Everglades National Park (ENP) is a large wetland surrounded on the north and east sides by developed areas in Miami-Dade County which are protected from flooding by levees and drainage canals bordering ENP. The large transmissivity of the underlying aquifer coupled with the operations of these canals results in large quantities of groundwater flowing from the wetlands to the east, resulting in potential flood control issues during periods of high water levels and/or rainfall.

To assess the impacts of water control strategies, ENP has developed a hydrologic model simulating both surface water and groundwater which covers approximately 1050 square miles of ENP and adjacent lands, as well as 110 miles of canals and associated structural components. These components include the detention areas adjacent to the South Dade Conveyance System (SDCS). The model is a combined MIKE SHE/MIKE 11 model, named the MIKE Marsh Model of ENP (M3ENP), which is capable of predicting hydrologic impacts of proposed structural and operational alternatives. In addition to evaluating wetland impacts, the model extends east into the developed areas and is thus able to determine potential flood impacts in the lands immediately east of the SDCS, along canals L-31N/C-111.

The M3ENP allows input of complex canal architecture and structure operational strategies. Along the northern and eastern boundaries of ENP the relevant canals, structures, and pumps are fully modeled and interact with both the surface water and groundwater regimes. The model grid contains 155 rows and 158 columns with a resolution of 400 meter square cells. Current simulations cover the time period from 1987 to 2005 using rainfall, potential evapotranspiration, and groundwater head boundary condition (except for a southern flux boundary) input at a daily resolution. The model outputs data for the canal system and overland flow with a 30 minute time step, and the unsaturated and saturated zones with time step of 2 and 12 hours, respectively.

The M3ENP model can provide the Central Everglades project a detailed evaluation of impacts on the ENP wetlands and adjacent developed areas under proposed alternative formulations. Structural and operational strategies considered along Tamiami Trail and the SDCS are not currently available but could be incorporated into the M3ENP. Due to these limitations, utilization of the model for CEPP may either (1) not be fully advocated by USACE; or (2) need to be applied with recognition of limitations, with results used only for a specific purpose and within a specific portion of the CEPP project area.

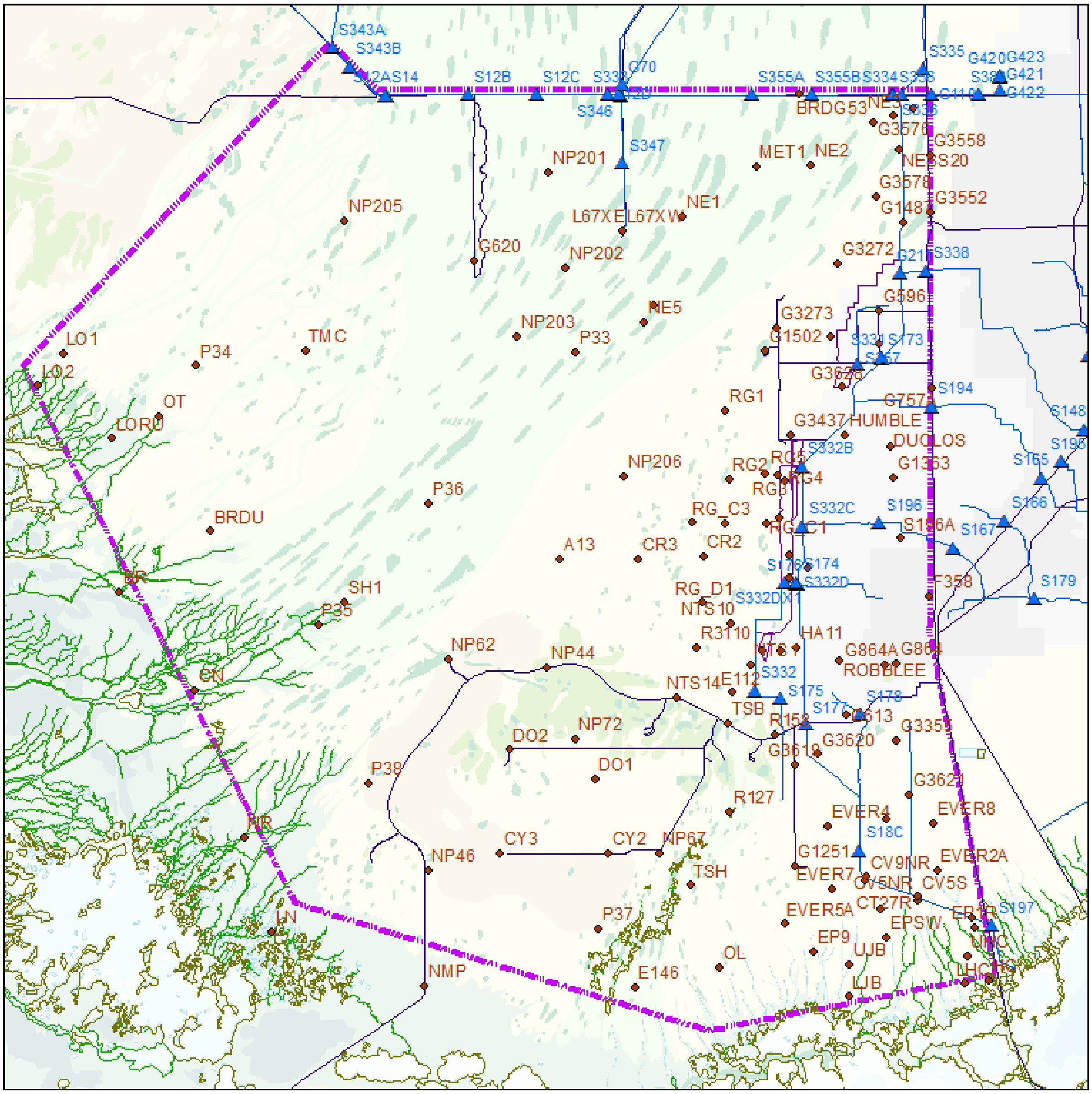


Figure 5.3.1. M3ENP MIKE-SHE/MIKE-11 model domain, showing monitoring points.

5.4 Everglades Landscape Model (ELM)

The DECOMP project plans on using the Everglades Landscape Model (ELM) to test the sensitivity of water quality loading on pristine areas for a range of potential water quality condiitons. However, for CEPP, project formulation will ensure that water quality standards will be met on all water deliveries across the red line. In addition, using ELM would be a challenge within the timeframe since it would need to be updated to adequately simulate the hydrology of alternative plans in order to represent the succession response of the landscape. For these reasons, use of ELM is not proposed for CEPP.

6.0 Model Performance Measures and Evaluation Tools

Performance measures are indicators of conditions in the natural system that have been determined to be characteristic of a healthy, restored ecosystem. Performance measures are used to predict performance of alternative plans. The CEPP team has identified a preliminary list of project performance measures to be used in the planning effort, reviewing performance measures used previously for CERP projects to quantify ecosystem benefits, as well as system-wide performance measures reviewed by RECOVER. The intent is to comprehensively evaluate all aspects of the system in a concise manner. **Table 6.1** lists a known subset of performance measures that are proposed for use. **Table 6.2** lists other indicators and evaluation tools that will be used to post-process model data. While other tools are likely to be used, these tables summarize the tools to be provided by the Modeling Team.

While this preliminary list of performance measures has been identified, evaluation strategies for the CEPP are still being developed, with the level of complexity and scope of evaluation remaining undefined as of the development of this strategy. As previously stated, the modeling tools described in this document will provide adequate hydrologic information to allow assessment of the identified performance measures to date.

Table 6.1. Performance Measures to be provided by HESM

Planning Region	Performance Measure / Evaluation Tool	Description	Original Model	Development Need
Lake Okeechobee	**Lake Stage** Extreme High and Low Water Levels in Greater Everglades Wetlands	Measure of the number and duration of extreme high and low water depth events.	NERSM	None identified
Northern Estuaries	**Salinity Envelopes** Oyster Habitat and Submerged Aquatic Vegetation	Measure of oyster and sea grass habitat based on frequency of flows from S-79 and S-80.	NERSM	None identified
Greater Everglades	**Hydrologic Surrogate for Soil Oxidation**	Measure of cumulative drought intensity to reduce exposure of peat to oxidation.	RSM DECOMP	41 yr extension
	Inundation Pattern in Greater Everglades Wetlands	Measure of the number and duration of inundation events used to calculate the percent period of record of inundation.	RSM DECOMP	41 yr extension
	Number and Duration of Dry Events in Shark River Slough	Measure of the number of times and mean duration in weeks that water level drops below ground.	SFWMM	Read RSM data
	Sheet flow in the Everglades Ridge and Slough Landscape	Measure of the timing and distribution of sheet flow across the landscape.	RSM DECOMP	41 yr extension
	Slough Vegetation Suitability	Measure to evaluate the hydrologic suitability for slough vegetation.	RSM DECOMP	41 yr extension

Table 6.2. Other Indicators and Evaluation Tools to be provided by HESM

Performance Measure / Evaluation Tool	Original Model	Development Need
Lake Okeechobee Minimum Flows & Levels Evaluation	NERSM	None identified
Lake Okeechobee Service Area Water Supply 4 in 1	NERSM	Use RSMBN data for eaa
Lake Okeechobee Service Area Water Supply Worst Years	NERSM	Use RSMBN data for EAA
Lower East Coast 1983-93 Window Performance Measure	SFWMM	Read RSM data
Everglades Viewing Windows	ALL	None identified
Hydrologic Maps (Flow Vector, Hydroperiod, Ponding, etc.)	RSM DECOMP	None identified
Everglades Restoration Transition Plan Performance Measures and Ecological Targets		Migrate from xls app to PMS
Cape Sable Seaside Sparrow	SFWMM	Read RSM data
Florida Bay Salinity	SFWMM	41 yr extension; Read RSM data; check on regression validities

7.0 Model Certification / Approval for Use

In parallel with the CEPP modeling analysis phase, the USACE processes for seeking model approval for use will be initiated on models that currently have not received the preferred designation. It is anticipated that staff from the USACE Jacksonville Engineering branch will work with the CEPP modeling team to generate and submit any necessary packages of information and facilitate working through any necessary responses as the process progresses.

8.0 Sources of Additional Information

The following sources represent a partial reference list:

Ali, A. 2009. Nonlinear multivariate rainfall-stage model for large wetland systems.. J of Hydrology. 374(2009)338-350.

Fan, A. 1986. A routing model for the upper Kissimmee chain of lakes. Tech. Pub 86-5. South Florida Water Management District, West Palm Beach, FL.

Neidrauer, Calvin J., Luis G. Cadavid, Paul J. Trimble, and Jayantha T.B. Obeysekera. 2006. A Spreadsheet-based Screening Model for Evaluating Alternative Water Management Strategies for Lake Okeechobee, Florida. Proceedings of the Operations Management 2006 Conference "Operating Reservoirs in Changing Conditions", Environmental Water Resources Institute (EWRI) of the American Society of Civil Engineers (ASCE). August 14-16, 2006, Sacramento, CA http://ascelibrary.org/proceedings/resource/2/ascecp/212/40875/35_1?isAuthorized=no

RSM Basins used in Northern Everglades Planning. http://my.sfwmd.gov/portal/page/portal/xrepository/sfwmd_repository_pdf/ne_crwpp_main_123108.pdf http://my.sfwmd.gov/portal/page/portal/xrepository/sfwmd_repository_pdf/ne_slrwpp_main_123108.pdf

Smajstrla, A.G. 1990. Agricultural Field Scale Irrigation Requirements Simulation (AFSIRS) Model. University of Florida Agricultural Engineering Technical Manual, Gainesville, FL.

South Florida Water Management District, 2005. Final Documentation for the South Florida Water Management Model. Hydrologic & Environmental Systems Modeling Department, Everglades Restoration, SFWMD, West Palm Beach, Florida.

South Florida Water Management District, 2010. DRAFT REPORT - Calibration and Validation of the Glades and Lower East Coast Service Area Application of the Regional Simulation Model, Sept 2010. Hydrologic & Environmental Systems Modeling Department, Everglades Restoration, SFWMD, West Palm Beach, Florida.

Starnes, Janet and Beth Marlowe. 2007. Spreadsheet Model and Water Budget Analysis for C-43 Project Delivery Team. Technical Memorandum of the Interagency Modeling Center (IMC). March 6, 2007. MSR 262.

Trimble, P.J., Obeysekera, J.T.B., Cadavid, L., and Santee, E.R. (2006). "Application of Climate Outlooks for Water Management in South Florida", in Climate Variations, Climate Change, and Water Resources Engineering, edited by J.D. Garbrecht and T.C. Piechota, ASCE.

U.S. Army Corps of Engineers, (1999b). "Integrated Feasibility Report and Environmental Impact Statement for Lake Okeechobee Regulation Schedule Study", Jacksonville District, Florida. http://planning.saj.usace.army.mil/envdocs/envdocsb.htm#Glades-County

United States Army Corps of Engineers – Hydrologic Engineering Center. http://www.hec.usace.army.mil/software/hec-ras/

Walker and Kadlec 2005, http://wwwalker.net/dmsta/

ANNEX A-2: REFERENCE 2

RSM-BN AND RSM-GL MODELING ASSUMPTIONS TABLES

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Basins (RSMBN) 2010 / 2011 Existing Conditions Baseline (ECB) Table of Assumptions

Feature	
Climate	• The climatic period of record is from 1965 to 2005 • Rainfall estimates have been revised and updated for 1965-2005 • Revised evapotranspiration methods have been used for 1965-2005
Topography	The Topography dataset for RSM was Updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004 • High Accuracy Elevation Data , US Geological Survey 2007 • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004 • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007 • Palm Beach County LiDAR Survey, Dewberry and Davis, 2004 • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/12, as reflected in the LOSA Ledger produced by the Water Use Bureau • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).

Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008)
	  o Includes Lake Okeechobee regulatory releases to tide via L8/C51 canals
	  o Lake Okeechobee regulatory releases limited to 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal based on studies performed by USACE.
	  o A regional hydrologic surrogate for the 2010 Adaptive Protocol operations utilized. This attempts to mimic desired timing of releases without estimating salinity criteria
	• Lake Okeechobee Water Shortage Management (LOWSM) Plan
	• Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and s-3 is to be minimized)
	• “Temporary” forward pumps as follows:
	  o S354 – 400 cfs
	  o S351 – 600 cfs
	  o S352 – 400 cfs
	  o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages

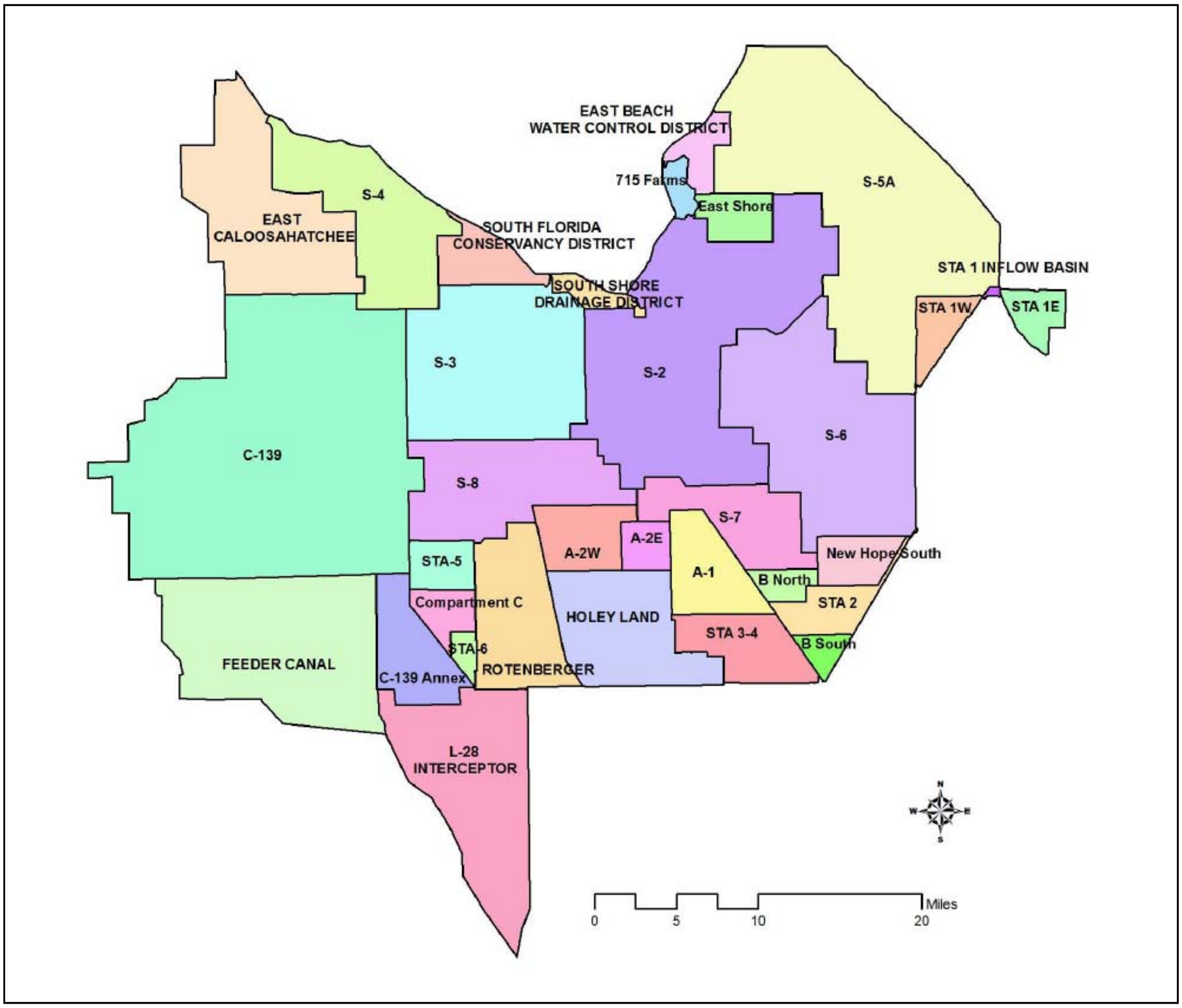
Feature	
	recover to greater than 11.2 ft. • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Operational intent is to treat LOK regulatory releases to the south through STA-3/4 • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Kissimmee River inflows based on interim schedule for Kissimmee Chain of Lakes using the UKISS model • Restored reaches / pools of Kissimmee River as of 2010 • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012 (see land use assumptions row). • Public water supply daily intake from the river is included in the analysis.
St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012(see land use assumptions row). • Excess C-44 basin runoff is allowed to backflow into the Lake if the lake stage is 0.25 ft below the Zone D pulse release line. • Basin demands include the Florida Power & Light reservoir at Indiantown.
Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage • The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons/month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved • LOWSM applies to this agreement
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved • LOWSM applies to this agreement

Feature	
Everglades Agricultural Area	• Model water-body components as shown in Figure 1 below. • Simulated runoff from the North New River – Hillsboro basin will be apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 routes water from S-5A Basin to Hillsboro Basin • EAA runoff and irrigation demand compared to SFWMM (ECB) simulated runoff and demand from 1965-2005 for reasonability • Compartment C land in the Miami Canal Basin between STA-5 and STA-6 is not considered to be in production (shrub Land Use). Then, no irrigation demands are required in this area. • Compartment B (excluding cell 4) land in the North New River/Hillsboro is not considered to be in production (shrub Land Use). Then, no irrigation demands are required in this area.

Stormwater Treatment Areas	• STAs are simulated as single waterbodies
	• STA-1E: 6,546 acres total area
	• STA-1W: 7,488 acres total area
	• S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E
	• STA-2: includes first four cells: 9,910 acres total area
	• STA-3/4: 17,126 acres total area
	• STA-5: includes first 3 cells: 7,619 acres total area
	• STA-6: 2,486 acres total area
	• Assumed operations of STAs:
	  • 0.5 ft minimum depth below which supply from external sources is triggered
	  • 4 ft maximum depth above which inflows are discontinued
	• STA-3/4 receives Lake Okeechobee regulatory releases approximately at 60,000 acre-feet annual average for the entire period of record.

Holey Land Wildlife Management Area	• G-372HL is the only inflow structure for Holey Land used for environmental purposes only • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FWC and the SFWMD
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA (SFWMD, March 2010)
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL ECB.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5; G406 flows routed to STA6 • C139 basin demand is met primarily by local groundwater
Water Shortage	• Reflects the existing water shortage policies as in South Florida

Feature	
Rules	Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan.



Water-Body Components:
Miami Water-Body = S3 + S8 + A-2W
NNR/HILLS Water-Body = S2 + S6 + S7 + A-2E + A-1 + B North + B South + New Hope South
WPB Water-Body = S-5A

Fig. 1 RSMBSN Basin Definition within the EAA: Existing Conditions Baseline Simulation

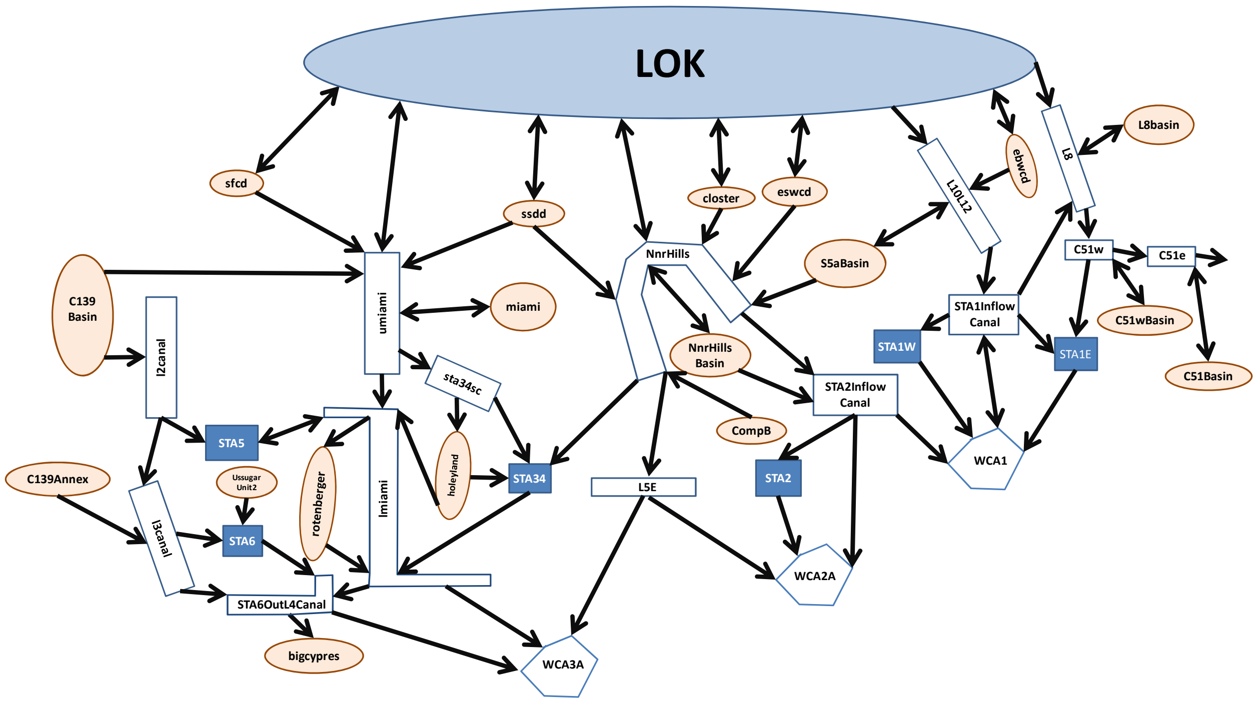


Fig. 2 RSMBSN Link-Node Routing Diagram: Existing Conditions Baseline Simulation

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the eastern and southern boundaries of the RSMBN model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Glades-LECSA Model (RSMGL). The SFWMM was the source of the eastern boundary groundwater/surface water flows, while the RSMGL was the source of the southern boundary structural flows.

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Glades-LECSA (RSMGL) 2010 / 2011 Existing Conditions Baseline (ECB) Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area. • Modified at locations where reservoirs are introduced (STA1-E, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed.
Lake Belt Lakes	• Based on 2005 Lake Belt Lake coverage obtained from USACE.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System

Feature	
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels in WCA-2A are less than minimum operating criteria of 10.5 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Current C&SF regulation schedule for WCA-3A, as per Water Control Plan –Interim Operational Plan (IOP) for protection of the Cape Sable seaside sparrow- C&SF Project for Flood Control and other Purposes (USACE, June 2006) • Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2006) • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Everglades Construction Project Stormwater Treatment Areas	• STA-1E: 5,132 acres total treatment area. • A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.
Everglades National Park	• Water deliveries to Everglades National Park are based upon the Interim Operational Plan (IOP) • L-29 stage constraint for operation of S-333 assumed to be 7.5 ft, NGVD. • G-3273 constraint for operation of S-333 assumed to be 6.8 ft, NGVD. • Tamiami Trail culverts east of the L67 Extension are simulated. • 5.5 miles remain of the L-67 Extension Levee. • S-355A & S-355B are operated. • S-356 is not operated. • Partial construction of C-111 project reservoirs consistent with the 2009 as-built information from USACE (does not include contract 8 or contract 9). A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed. • S-332DX1 is not operated. • 8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2.
Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay

Feature	
Pumpage and Irrigation	• Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included • Residential Self Supported (RSS) pumpage are based on 2030 projections from the SFWMD Water Supply Bureau. • Industrial pumpage are based on 2030 projections from the SFWMD Water Supply Bureau. • Irrigation demands for the six irrigation land-use types are calculated internally by the model. • Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.
Canal Operations	• C&SF system and operating rules in effect in 2010 • Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion • Includes existing secondary drainage/water supply system • C-4 Flood Mitigation Project • Western C-4, S-380 structure retained open • C-11 Water Quality Treatment Critical Project (S-381 and S-9A) • S-25B and S-26 pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary • Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry • ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure • Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Interim Operational Plan (IOP): o Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15 o Structure S-12B is closed Jan. 1 to July 15 o Structure S-12C is closed Feb. 1 to July 15 • South Dade Conveyance System operations will follow IOP for protection of the Cape Sable seaside sparrow
Canal Configuration	• Canal configuration same as calibration except only 5.5 miles remain of the L-67 Extension Canal.
Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data. • Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the corresponding RSMBN ECB simulation.

Notes

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Basins (RSMBN)
2012 Existing Conditions (2012EC) Baseline
Table of Assumptions

Note: RSMBN CEPP 2012EC (2/28/13) is identical to the RSMBN CEPP ECB (12/13/12)

Feature	
Climate	• The climatic period of record is from 1965 to 2005 • Rainfall estimates have been revised and updated for 1965-2005 • Revised evapotranspiration methods have been used for 1965-2005
Topography	The Topography dataset for RSM was Updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004 • High Accuracy Elevation Data , US Geological Survey 2007 • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004 • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007 • Palm Beach County LiDAR Survey, Dewberry and Davis, 2004 • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/12, as reflected in the LOSA Ledger produced by the Water Use Bureau • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).

Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008)
	  o Includes Lake Okeechobee regulatory releases to tide via L8/C51 canals
	  o Lake Okeechobee regulatory releases limited to 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal based on studies performed by USACE.
	  o A regional hydrologic surrogate for the 2010 Adaptive Protocol operations utilized. This attempts to mimic desired timing of releases without estimating salinity criteria
	• Lake Okeechobee Water Shortage Management (LOWSM) Plan
	• Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and S-3 is to be minimized)
	• "Temporary" forward pumps as follows:
	  o S354 – 400 cfs
	  o S351 – 600 cfs
	  o S352 – 400 cfs

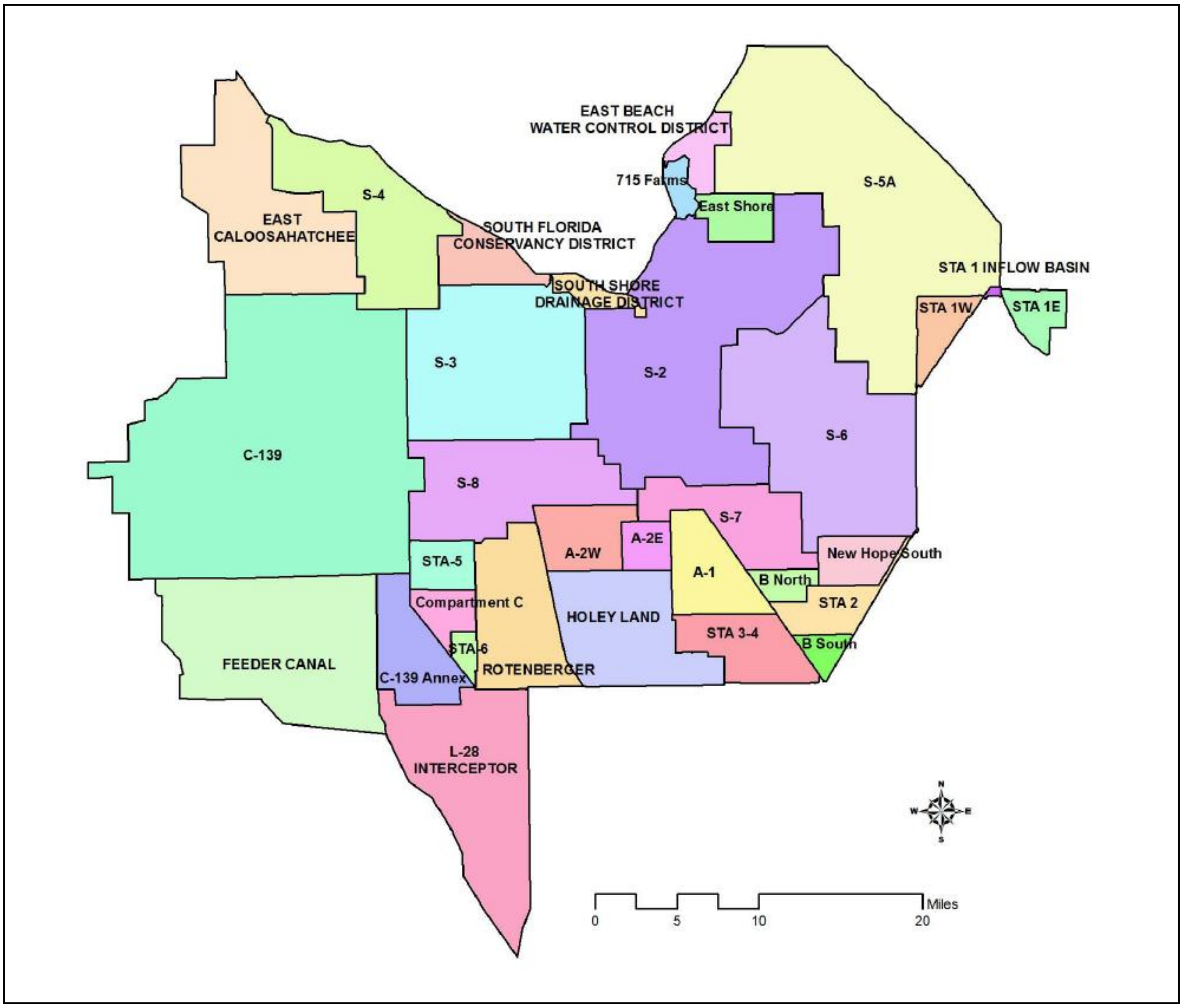
Feature	
	○ All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages recover to greater than 11.2 ft. • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Operational intent is to treat LOK regulatory releases to the south through STA-3/4 • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Kissimmee River inflows based on interim schedule for Kissimmee Chain of Lakes using the UKISS model • Restored reaches / pools of Kissimmee River as of 2010 • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012 (see land use assumptions row). • Public water supply daily intake from the river is included in the analysis.
St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012(see land use assumptions row). • Excess C-44 basin runoff is allowed to backflow into the Lake if the lake stage is 0.25 ft below the Zone D pulse release line. • Basin demands include the Florida Power & Light reservoir at Indiantown.
Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage • The 2-in10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons per month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved • LOWSM applies to this agreement
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved • LOWSM applies to this agreement

Feature	
Everglades Agricultural Area	• Model water-body components as shown in Figure 1 below. • Simulated runoff from the North New River – Hillsboro basin will be apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 routes water from S-5A Basin to Hillsboro Basin • EAA runoff and irrigation demand compared to SFWMM (ECB) simulated runoff and demand from 1965-2005 for reasonability • Compartment C land in the Miami Canal Basin between STA-5 and STA-6 is not considered to be in production (shrub Land Use). Then, no irrigation demands are required in this area. • Compartment B (excluding cell 4) land in the North New River/Hillsboro is not considered to be in production (shrub Land Use). Then, no irrigation demands are required in this area.

Stormwater Treatment Areas	• STAs are simulated as single waterbodies
	• STA-1E: 6,546 acres total area
	• STA-1W: 7,488 acres total area
	• S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E
	• STA-2: includes first four cells: 9,910 acres total area
	• STA-3/4: 17,126 acres total area
	• STA-5: includes first 3 cells: 7,619 acres total area
	• STA-6: 2,486 acres total area
	• Assumed operations of STAs:
	  o 0.5 ft minimum depth below which supply from external sources is triggered
	  o 4 ft maximum depth above which inflows are discontinued
	• STA-3/4 receives Lake Okeechobee regulatory releases approximately at 60,000 acre-feet annual average for the entire period of record.

Holey Land Wildlife Management Area	• G-372HL is the only inflow structure for Holey Land used for environmental purposes only • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FWC and the SFWMD
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA (SFWMD, March 2010)
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL ECB.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5; G406 flows routed to STA6 • C139 basin demand is met primarily by local groundwater

Feature	
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan.



Water-Body Components:

Miami Water-Body = S3 + S8 + A-2W

NNR/HILLS Water-Body = S2 + S6 + S7 + A-2E + B North + B South + New Hope South

WPB Water-Body = S-5A

Fig. 1 RSMBSN Basin Definition within the EAA: 2012 Existing Conditions Simulation

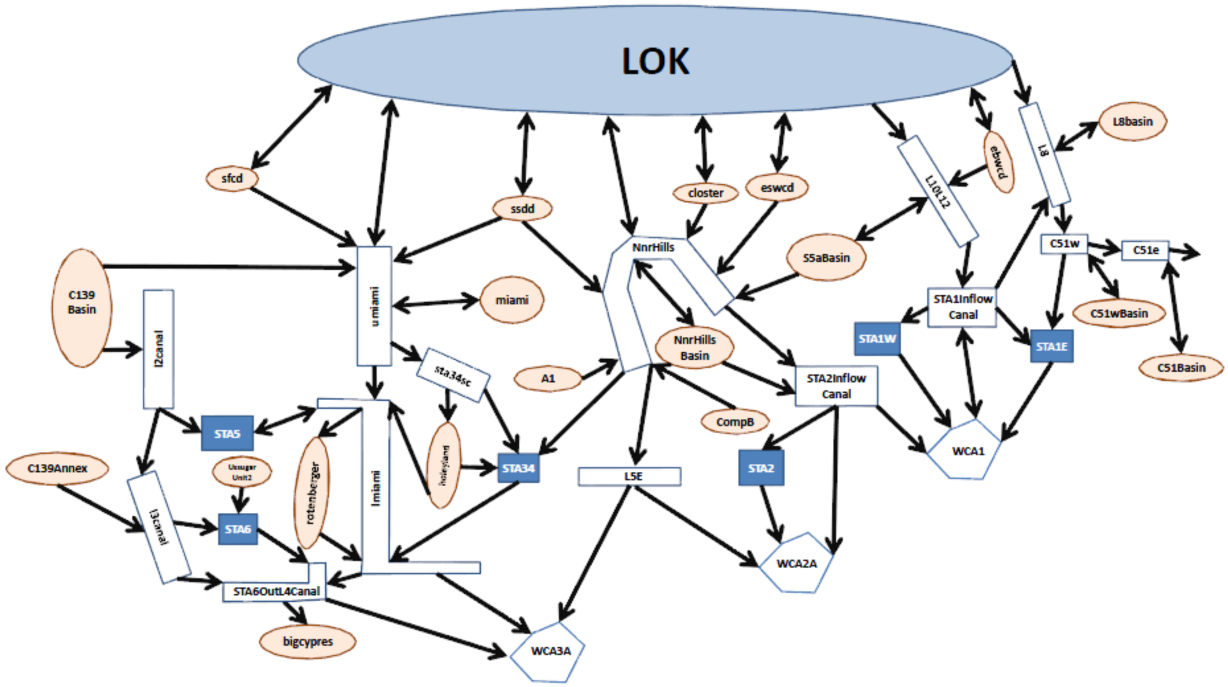


Fig. 2 RSMBSN Link-Node Routing Diagram: 2012 Existing Conditions Simulation

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the eastern and southern boundaries of the RSMBN model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Glades-LECSA Model (RSMGL). The SFWMM was the source of the eastern boundary groundwater/surface water flows, while the RSMGL was the source of the southern boundary structural flows.
- 2012EC assumptions were updated from the CEPP 12/13/2012 ECB scenario at the time that the CEPP tentatively selected plan was identified.

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Glades-LECSA (RSMGL) 2012 Existing Conditions (2012EC) Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area. • Modified at locations where reservoirs are introduced (STA1-E, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed.
Lake Belt Lakes	• Based on 2005 Lake Belt Lake coverage obtained from USACE.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System

Feature	
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels in WCA-2A are less than minimum operating criteria of 10.5 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 (USACE, 2012). • Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2006) • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Everglades Construction Project Stormwater Treatment Areas	• STA-1E: 5,132 acres total treatment area. • A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.
Everglades National Park	• Water deliveries to Everglades National Park are based upon Everglades Restoration Transition Plan (ERTP), with the WCA-3A Regulation Schedule including the lowered Zone A (compared to IOP) and extended Zones D and E1. • L-29 stage constraint for operation of S-333 assumed to be 7.5 ft, NGVD. • G-3273 constraint for operation of S-333 assumed to be 6.8 ft, NGVD. • Tamiami Trail culverts east of the L67 Extension are simulated. • 5.5 miles remain of the L-67 Extension Levee. • S-355A & S-355B are operated. • S-356 is not operated. • Partial construction of C-111 project reservoirs consistent with the 2009 as-built information from USACE (does not include contract 8 or contract 9). A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed. • S-332DX1 is not operated. • 8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2.
Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay

Feature	
Pumpage and Irrigation	• Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included • Residential Self Supported (RSS) pumpage are based on 2030 projections from the SFWMD Water Supply Bureau. • Industrial pumpage are based on 2030 projections from the SFWMD Water Supply Bureau. • Irrigation demands for the six irrigation land-use types are calculated internally by the model. • Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.

Canal Operations	• C&SF system and operating rules in effect in 2012
	• Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion
	• Includes existing secondary drainage/water supply system
	• C-4 Flood Mitigation Project
	• Western C-4, S-380 structure retained open
	• C-11 Water Quality Treatment Critical Project (S-381 and S-9A).
	   o S9/S9A operations modified for performance consistency with SFWMM ECB.
	• S-25B and S-26 pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary
	• Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry
	• ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure
	• Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1
	   o Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15
	   o Structure S-12B is closed Jan. 1 to July 15
	• South Dade Conveyance System operations will follow ERTP for protection of the Cape Sable seaside sparrow

Canal Configuration	• Canal configuration same as calibration except only 5.5 miles remain of the L-67 Extension Canal.
Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data.

Feature	
	• Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the corresponding RSMBN ECB simulation.

Notes

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.
- 2012EC assumptions were updated from the CEPP 12/13/2012 ECB scenario at the time that the CEPP tentatively selected plan was identified.

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Basins (RSMBN) 2050 Future Without Project Baseline (FWO) Table of Assumptions

Feature	
Climate	• The climatic period of record is from 1965 to 2005 • Rainfall estimates have been revised and updated for 1965-2005 • Revised evapotranspiration methods have been used for 1965-2005
Topography	The Topography dataset for RSM was Updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004 • High Accuracy Elevation Data , US Geological Survey 2007 • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004 • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007 • Palm Beach County LiDAR Survey, Dewberry and Davis, 2004 • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/2012, as reflected in the LOSA Ledger produced by the Water Use Bureau. • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).

Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008)
	  o Includes Lake Okeechobee regulatory releases to tide via L8/C51 canals
	  o Lake Okeechobee regulatory releases limited to 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal based on studies performed by USACE.
	  o Releases via S-77 can be diverted into C43 Reservoir
	• No Lake Okeechobee environmental releases.
	• Lake Okeechobee Water Shortage Management (LOWSM) Plan
	• Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and s-3 is to be minimized)
	• “Temporary” forward pumps as follows:
	  o S354 – 400 cfs
	  o S351 – 600 cfs
	  o S352 – 400 cfs
	  o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages

Feature	
	recover to greater than 11.2 ft. • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Operational intent is to treat LOK regulatory releases to the south through STA-3/4 • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Headwaters Revitalization schedule for Kissimmee Chain of Lakes using the UKISS model • Kissimmee River Restoration complete. • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012. (see land use assumptions row) • Public water supply daily intake from the river is included in the analysis. • Maximum reservoir height of 41.7 ft NGVD with a 9,379-acre footprint in Western C43 basin with a 175,800 acre-feet effective storage. • Proposed reservoir meets estuary demands while C-43 basin supplemental demands for surface water irrigation are met by Lake Okeechobee.

St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012(see land use assumptions row).
	• Excess C-44 basin runoff is allowed to backflow into the Lake if lake stage is 0.25 ft. below the Zone D pulse release line before being pumped into the C-44 reservoir.
	• Basin demands include the Florida Power & Light reservoir at Indiantown.
	• Indian River Lagoon South Project features
	  • Ten-mile Creek Reservoir and STA: 7,078 acre-feet storage capacity at 10.79 maximum depth on 820 acre footprint; receives excess water from North Folk Basin
	  • C-44 reservoir: 50,246 acre-feet storage capacity at 5.18 feet maximum depth on 12,125 acre footprint
	  • C-23/C-24 reservoir: 92,094 acre-feet storage capacity at 13.27 maximum depth on 8,675 acre footprint
	  • C-23/C-24 STA: 3,852 acre-feet storage capacity at 1.5 maximum depth on 2,568 acre footprint
	  • All proposed reservoirs meet estuary demands

Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons/month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal

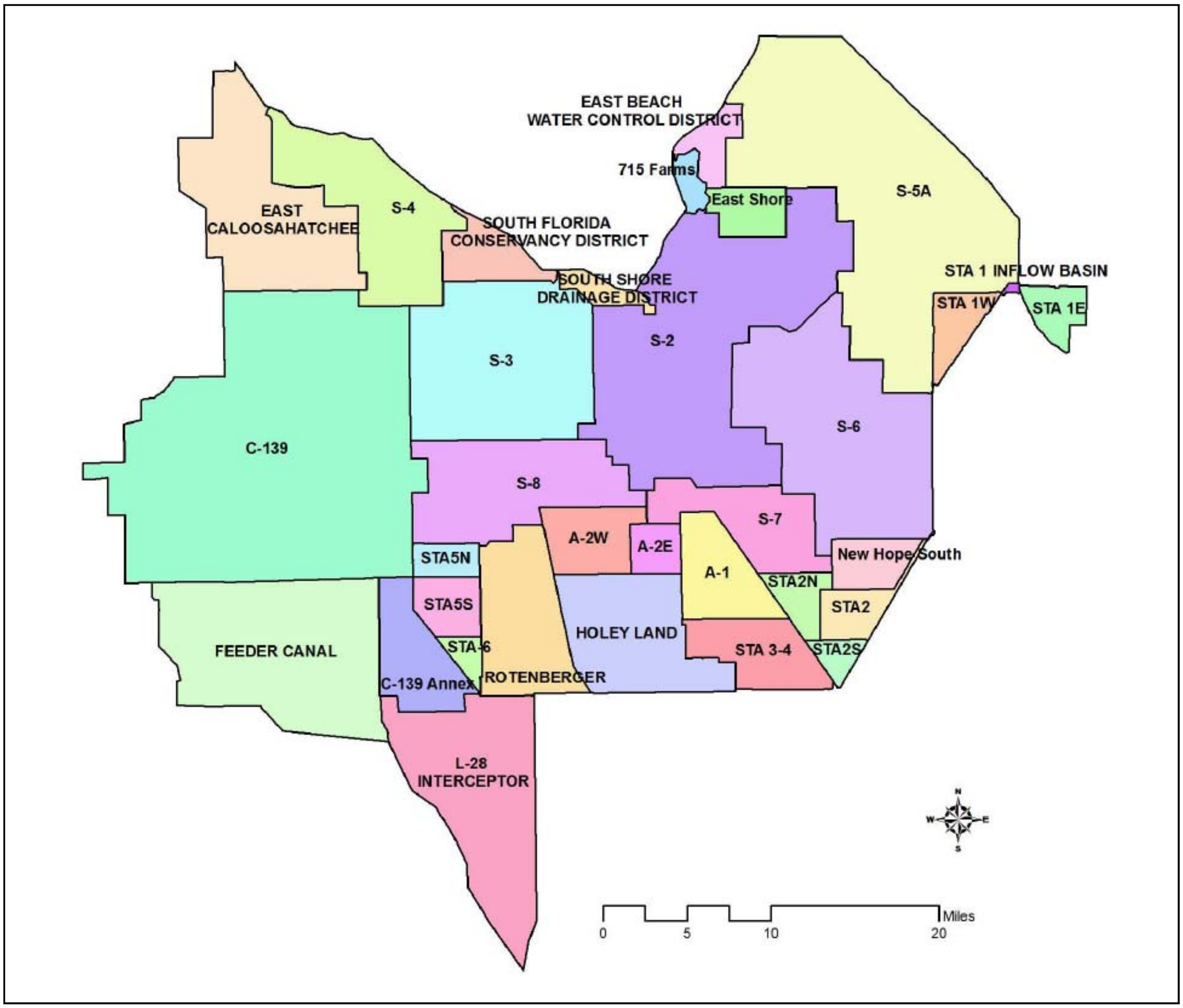
Feature	
	rights to these quantities are preserved • LOWSM applies to this agreement
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved • LOWSM applies to this agreement
Everglades Agricultural Area	• Model water-body components as shown in Figure 1. • Simulated runoff from the North New River – Hillsboro basin apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 routes water from S-5A Basin to Hillsboro Basin. • RSMBN ECB EAA runoff and irrigation demand compared to SFWMM ECB simulated runoff and demand from 1965-2005 for reasonability.

Everglades Construction Project Stormwater Treatment Areas	• STAs are simulated as single waterbodies
	• STA-1E: 6,546 acres total area
	• STA-1W: 7,488 acres total area
	• S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E
	• STA-2: cells 1,2 & 3: 7,681 acres total area
	• STA-2N: cells 4,5 & 6; refers to Comp B-North; 6,531 acres total area
	• STA-2S: cells 7 & 8; refers to Comp B-South; 3,570 acres total area
	• STA-3/4: 17,126 acres total area
	• STA-5N: includes cells 1 & 2: 5,081 acres total area
	• STA-5S: includes cells 3, 4 & 5; uses footprint of Compartment C: 8,469 acres total area
	• STA-6: expanded with phase 2: 3,054 acres total area
	• Assumed operations of STAs:
	  • 0.5 ft minimum depth below which supply from external sources is triggered
	  • 4 ft maximum depth above which inflows are discontinued
	  • Inflow targets established for STA-3/4, STA-2N and STA-2S based on DMSTA simulation; met from local basin runoff, LOK regulatory discharge and available A1FEB storage.
	  • STA-3/4 receives Lake Okeechobee regulation target releases approximately at 60,000 acre-feet annual average for the entire

Feature	

	period of record.
	• A 15,853-acre Flow Equalization Basin (FEB) located north of STA-3/4.
	• Assumed operations of A1FEB:
	  • FEB inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south
	  • FEB outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, and STA-2S if EAA basin runoff and LOK regulatory discharge are not sufficient.
	  • 0.5 ft minimum depth below which no releases are allowed
	  • 3.8 ft maximum depth above which inflows are discontinued
	  • Assumed inlet pump from STA-3/4 supply canal with capacity equal to combined capacity of G-372 and G-370 structures.
	  • Outflow weirs, with similar discharge characteristics as STA-3/4 outlet structure, discharging into lower North New River canal.

Holey Land Wildlife Management Area	• G-372HL is the only inflow structure for Holey Land used for environmental purposes only • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FWC and the SFWMD
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA (SFWMD, March 2010)
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL FWO.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5N; G508 flows routed to STA5S; G406 flows routed to STA6C139 basin demand is met primarily by local groundwater
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan



Water-Body Components:
Miami Water-Body = S3 + S8
NNR/HILLS Water-Body = S2 + S6 + S7 + A-2E + New Hope South
WPB Water-Body = S-5A
A1FEB = A-1

Fig. 1 RSMBSN Basin Definition within the EAA: Future Without Project Baseline Simulation

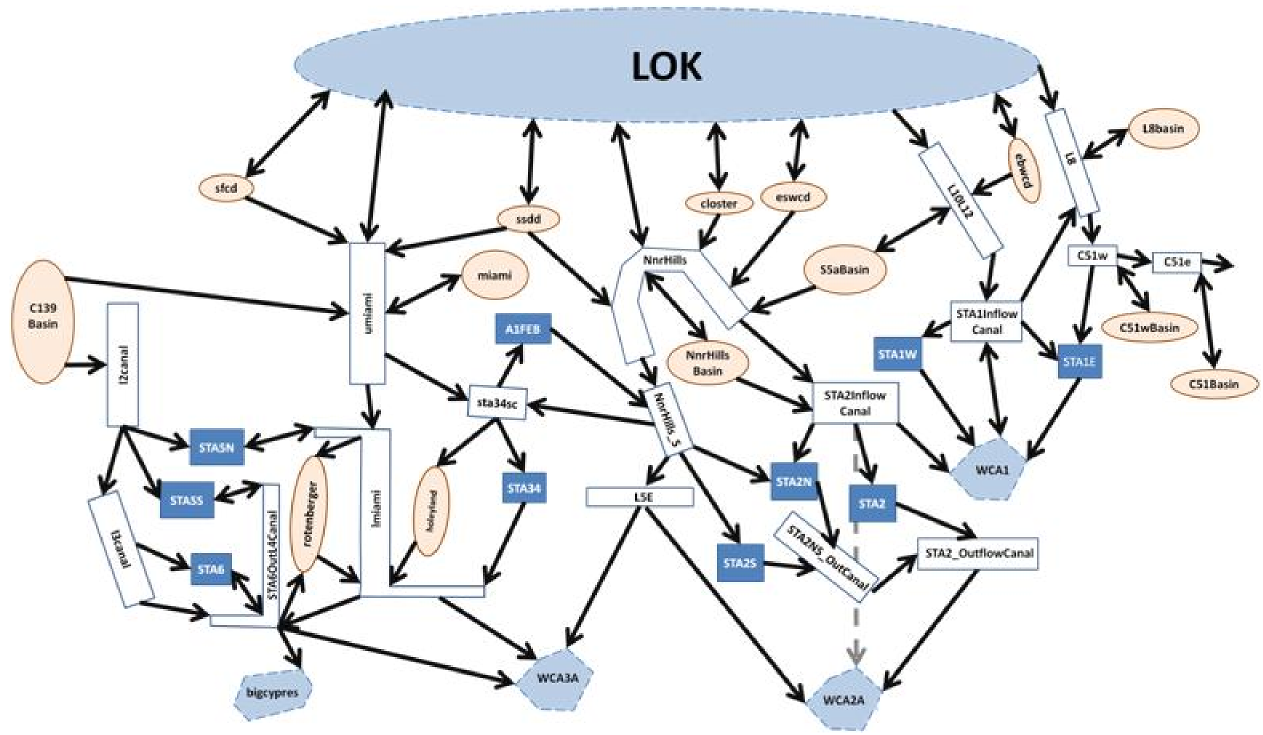


Fig. 2 RSMBSN Link-Node Routing Diagram: Future Without Project Baseline Simulation

Note:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
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Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Glades-LECSA (RSMGL) 2050 Future Without Project Baseline (FWO) Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area. • Modified at locations where reservoirs are introduced (STA1-E, Site 1 Impoundment, Broward WPAs, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed. • 8.5 SMA seepage canal is modeled as a WCD in ENP area.
Lake Belt Lakes	• Based on the permitted 2020 Lake Belt Lakes coverage obtained from USACE.
CERP Projects	• 1st Generation CERP – Site 1 Impoundment project is modeled as an above ground reservoir of area 1600 acres, with a maximum depth of 8 ft. • 2nd Generation CERP – Broward County Water Preserve Areas (WPAs) comprised of C-11 and C-9 impoundments were modeled as above ground reservoirs with areas 1221 and 1971 acres and maximum depths 4.3 and 4.0 ft. respectively.

Feature	
	• 2nd Generation CERP – C-111 Spreader Canal Project includes the Frog Pond Detention Area, which is modeled as an above ground impoundment with the S200 A, B and C pumps as inflow structures. In addition, the Aerojet canal is modeled with the inflow pumps S199 A, B and C. The S199 and S200 pumps are turned off based on the stage at the remote monitoring location EVER4 for the protection of the CSS Critical Habitat Unit 3. • 2nd Generation CERP – Biscayne Bay Coastal Wetlands project features were not modeled since these features along the coast in Miami-Dade County were not considered significant for CEPP. • Areal corrections were applied to the impoundment storages to account for the discrepancies of the areas in the model of the impoundments not matching the design areas.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels in WCA-2A are less than minimum operating criteria of 10.5 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 (USACE, 2012) • Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2002) • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Everglades Construction Project Stormwater Treatment Areas	• STA-1E: 5,132 acres total treatment area. • A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.

Feature	
Everglades National Park	• Water deliveries to Everglades National Park are based upon Everglades Restoration Transition Plan (ERTP), with the WCA-3A Regulation Schedule including the lowered Zone A (compared to IOP) and extended Zones D and E1. • L-29 stage constraint for operation of S-333 assumed to be 7.5 ft, NGVD. • G-3273 constraint for operation of S-333 assumed to be 6.8 ft, NGVD. • The one mile Tamiami Trail Bridge as per the 2008 Tamiami Trail Limited Reevaluation Report is modeled as a one mile weir. Located east of the L67 extension and west of the S334 structure. • Tamiami Trail culverts east of the L67 Extension are simulated where the bridge is not located. • 5.5 miles remain of the L-67 Extension Levee. • S-355A & S-355B are operated. • S-356 is not operated. • Full construction of C-111 project reservoirs consistent with the as-built information from USACE plus addition of contract 8 and contract 9 features. A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed. • 8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2. Outflow assumed from 8.5 SMA detention cell to the C-111 North Detention Area.
Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay
Pumpage and Irrigation	• Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included • Residential Self Supported (RSS) pumpage are based on 2010 projections of residential population from the SFWMD Water Supply Bureau. • Industrial pumpage is based on 2010 permits. • Irrigation demands for the six irrigation land-use types are calculated internally by the model. • Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.
Canal Operations	• C&SF system and operating rules in effect in 2010 • Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion • Includes existing secondary drainage/water supply system • C-4 Flood Mitigation Project • Western C-4, S-380 structure retained open

<table>
<tr><th>Feature</th><th></th></tr>
<tr><td></td><td>

C-11 Water Quality Treatment Critical Project (S-381 and S-9A)
S-25B and S-26 backflow pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary
Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry
ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure
Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1

Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15
Structure S-12B is closed Jan. 1 to July 15

</td></tr>
<tr><td>Canal Configuration</td><td>

Canal configuration same as calibration except only 5.5 miles remain of the L-67 Extension Canal and CERP project modifications.
</td></tr>
<tr><td>Lower East Coast Service Area Water Shortage Management</td><td>

Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data.
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- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.

`Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Basins (RSMBN)
Initial Operating Regime Baseline 1 (IORBL1)
Table of Assumptions

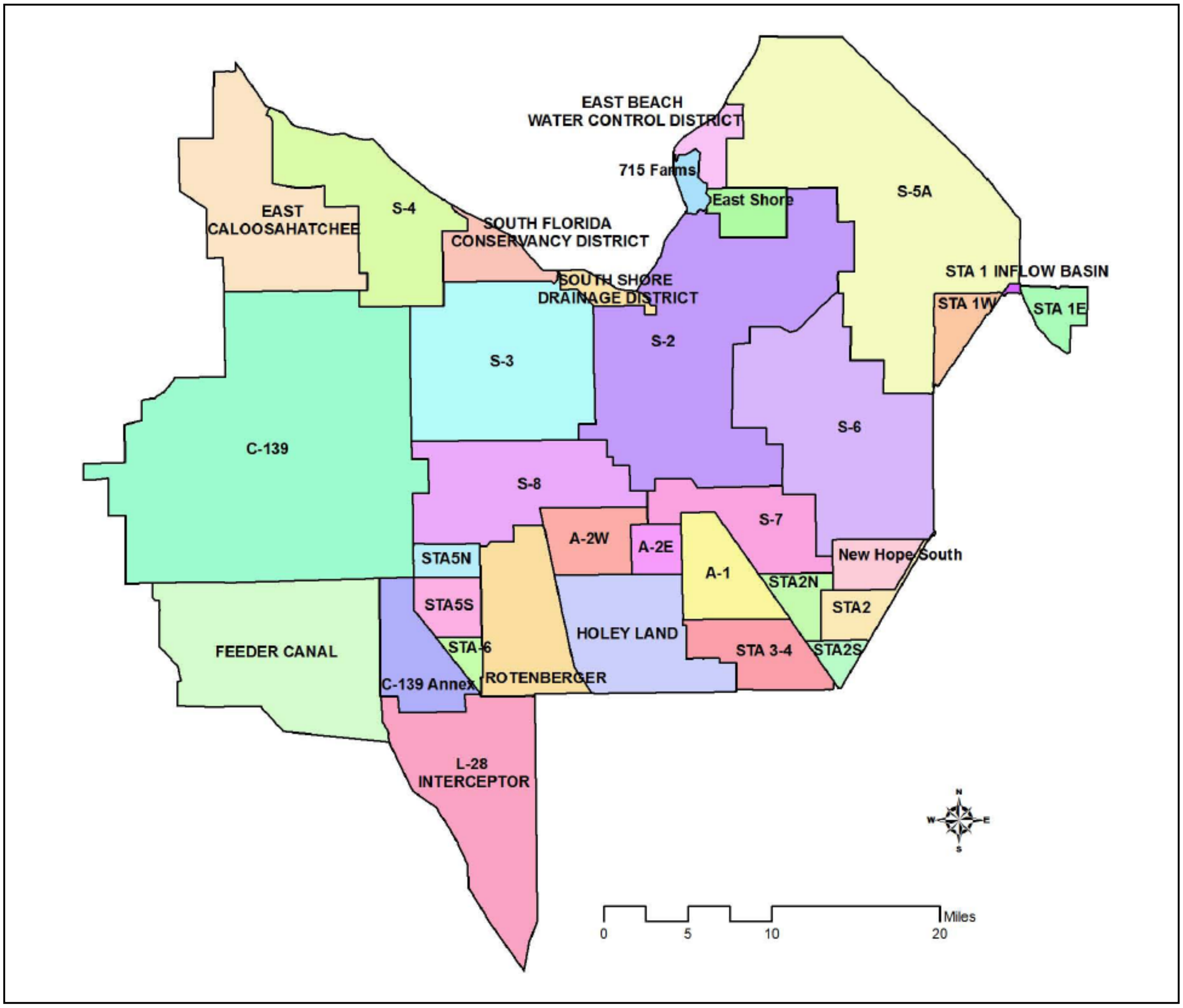
Feature	
Climate	• The climatic period of record is from 1965 to 2005 • Rainfall estimates have been revised and updated for 1965-2005 • Revised evapotranspiration methods have been used for 1965-2005
Topography	The topography dataset for RSM was updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004 • High Accuracy Elevation Data , US Geological Survey 2007 • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004 • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007 • Palm Beach County LiDAR Survey, Dewberry and Davis, 2004 • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/2012, as reflected in the LOSA Ledger produced by the Water Use Bureau. • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).

Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008)
	  o Includes Lake Okeechobee regulatory releases to tide via L8/C51 canals
	  o Lake Okeechobee regulatory releases limited to 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal based on studies performed by USACE.
	  o Releases via S-77 can be diverted into C43 Reservoir
	• No Lake Okeechobee environmental releases.
	• Lake Okeechobee Water Shortage Management (LOWSM) Plan
	• Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and S-3 is to be minimized)
	• "Temporary" forward pumps as follows:
	  o S354 – 400 cfs
	  o S351 – 600 cfs
	  o S352 – 400 cfs

Feature	
	o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages recover to greater than 11.2 ft. • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Operational intent is to treat LOK regulatory releases to the south through STA-3/4 • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Headwaters Revitalization schedule for Kissimmee Chain of Lakes using the UKISS model • Kissimmee River Restoration complete. • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012. (see land use assumptions row) • Public water supply daily intake from the river is included in the analysis. • Maximum reservoir height of 41.7 ft NGVD with a 9,379-acre footprint in Western C43 basin with a 175,800 acre-feet effective storage. • Proposed reservoir meets estuary demands while C-43 basin supplemental demands for surface water irrigation are met by Lake Okeechobee.
St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012(see land use assumptions row). • Excess C-44 basin runoff is allowed to backflow into the Lake if lake stage is 0.25 ft. below the Zone D pulse release line before being pumped into the C-44 reservoir. • Basin demands include the Florida Power & Light reservoir at Indiantown. • Indian River Lagoon South Project features o Ten-mile Creek Reservoir and STA: 7,078 acre-feet storage capacity at 10.79 maximum depth on 820 acre footprint; receives excess water from North Folk Basin o C-44 reservoir: 50,246 acre-feet storage capacity at 5.18 feet maximum depth on 12,125 acre footprint o C-23/C-24 reservoir: 92,094 acre-feet storage capacity at 13.27 maximum depth on 8,675 acre footprint o C-23/C-24 STA: 3,852 acre-feet storage capacity at 1.5 maximum depth on 2,568 acre footprint o All proposed reservoirs meet estuary demands o IRL operations assumed are consistent with the March 2010 St. Lucie River Water Reservation Rule update. o Excess C23 basin water not needed to meet estuary demands can be diverted to the C44 reservoir if capacity exists.
Seminole Brighton	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage.

Feature	
Reservation	• The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons per month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved • LOWSM applies to this agreement
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved • LOWSM applies to this agreement
Everglades Agricultural Area	• Model water-body components as shown in Figure 1. • Simulated runoff from the North New River – Hillsboro basin apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 routes water from S-5A Basin to Hillsboro Basin. • RSMBN ECB EAA runoff and irrigation demand compared to SFWMM ECB simulated runoff and demand from 1965-2005 for reasonability.
Everglades Construction Project Stormwater Treatment Areas	• STAs are simulated as single waterbodies • STA-1E: 6,546 acres total area • STA-1W: 7,488 acres total area • S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E • STA-2: cells 1,2 & 3: 7,681 acres total area • STA-2N: cells 4,5 & 6; refers to Comp B-North; 6,531 acres total area • STA-2S: cells 7 & 8; refers to Comp B-South; 3,570 acres total area • STA-3/4: 17,126 acres total area • STA-5N: includes cells 1 & 2: 5,081 acres total area • STA-5S: includes cells 3, 4 & 5; uses footprint of Compartment C: 8,469 acres total area • STA-6: expanded with phase 2: 3,054 acres total area • Assumed operations of STAs: o 0.5 ft minimum depth below which supply from external sources is triggered

Feature	
	o 4 ft maximum depth above which inflows are discontinued o Inflow targets established for STA-3/4, STA-2N and STA-2S based on DMSTA simulation; met from local basin runoff, LOK regulatory discharge and available A1FEB storage. o STA-3/4 receives Lake Okeechobee regulation target releases approximately at 60,000 acre-feet annual average for the entire period of record. • A 15,853-acre Flow Equalization Basin (FEB) located north of STA-3/4 with assumed operations as follows: o FEB inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south. o FEB outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, and STA-2S if EAA basin runoff and LOK regulatory discharge are not sufficient. o 0.5 ft minimum depth below which no releases are allowed o 3.8 ft maximum depth above which inflows are discontinued o Assumed inlet pump from STA-3/4 supply canal with capacity equal to combined capacity of G-372 and G-370 structures. o Outflow weirs, with similar discharge characteristics as STA-3/4 outlet structure, discharging into lower North New River canal. o Structure capacities and water quality operating rules are consistent with modeling assumptions assumed during the A-1 FEB EIS application process.
Holey Land Wildlife Management Area	• G-372HL is the only inflow structure for Holey Land used for environmental purposes only • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FWC and the SFWMD
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA (SFWMD, March 2010)
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5N; G508 flows routed to STA5S; G406 flows routed to STA6C139 basin demand is met primarily by local groundwater
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan



Water-Body Components:
Miami Water-Body = S3 + S8 + A-2W
NNR/HILLS Water-Body = S2 + S6 + S7 + A-2E + New Hope South
WPB Water-Body = S-5A
A1FEB = A-1

Fig. 1 RSMBSN Basin Definition within the EAA: Initial Operating Regime Baseline Simulation

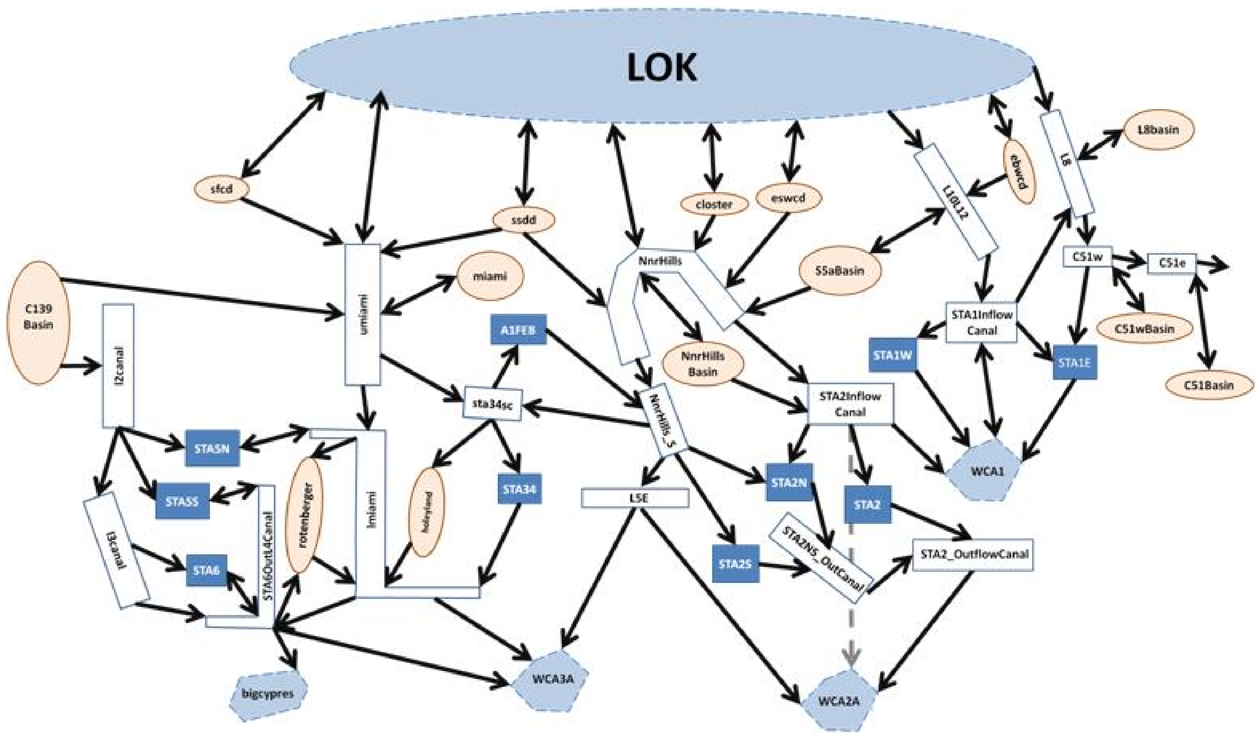


Fig. 2 RSMBSN Link-Node Routing Diagram: Initial Operating Regime Baseline Simulation

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- IORBL assumptions were updated from the CEPP 12/13/2012 FWO scenario at the time that the CEPP tentatively selected plan was identified and then adjusted for the IRL project to produce the IORBL1.

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Glades-LECSA (RSMGL) Initial Operating Regime Baseline 1 (IORBL1) Table of Assumptions

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Feature	
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Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay
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Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data. • Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the RSMBN FWO simulation.

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).

- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.
- IORBL assumptions were updated from the CEPP 12/13/2012 FWO scenario at the time that the CEPP tentatively selected plan was identified and then adjusted for the Broward County WPA project to produce the IORBL1.

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Basins (RSMBN) Alternative 4 Revised (ALT4R) Scenario Table of Assumptions

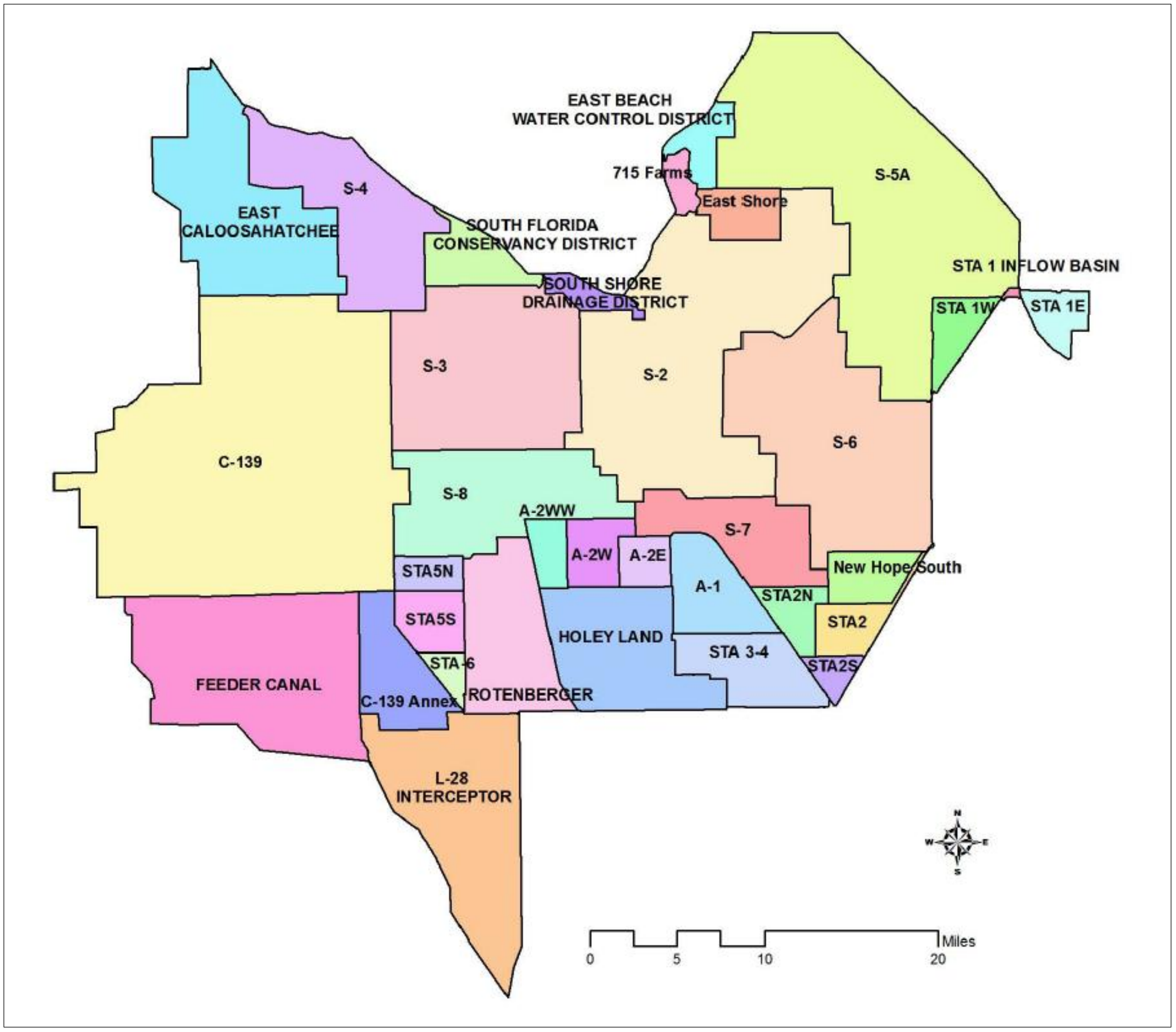
Feature	
Climate	• The climatic period of record is from 1965 to 2005. • Rainfall estimates have been revised and updated for 1965-2005. • Revised evapotranspiration methods have been used for 1965-2005.
Topography	The Topography dataset for RSM was Updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004; • High Accuracy Elevation Data, US Geological Survey 2007; • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004; • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007; • Palm Beach County LiDAR Surve, Dewberry and Davis, 2004; and • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/2012, as reflected in the LOSA Ledger produced by the Water Use Bureau. • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information . • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass.
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).
Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008) o CEPP optimized release guidance in order to improve selected performance within LOK, the northern estuaries and LOSA while meeting environmental targets in the Glades. o Lake Okeechobee can send flood releases south through the Miami Canal and North New River Canal to the FEB when the LOK stage is above the bottom of Zone D and the FEB depth is below 2’ (EAA basin runoff used to limit conveyance capacity: 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal). o Lake Okeechobee can send flood releases south to help meet water-quality based flow targets at STA-3/4, STA-2N, and STA-2S when the LOK stage is above the bottom of the Baseflow Zone (EAA basin runoff used to limit conveyance capacity: 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal). o Includes Lake Okeechobee regulatory releases to tide via L8 canal.

Feature	
	o Releases via S-77 can be diverted into C43 Reservoir • Lake Okeechobee Water Shortage Management (LOWSM) Plan. • Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and S-3 is to be minimized). • "Temporary" forward pumps as follows: o S354 – 400 cfs o S351 – 600 cfs o S352 – 400 cfs o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages recover to greater than 11.2 ft • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Headwaters Revitalization schedule for Kissimmee Chain of Lakes using the UKISS model. • Kissimmee River Restoration complete. • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012. (see land use assumptions row) • Public water supply daily intake from the river is included in the analysis. • Maximum reservoir height of 41.7 ft NGVD with a 9,379-acre footprint in Western C43 basin with a 175,800 acre-feet effective storage. • Proposed reservoir meets estuary demands while C-43 basin supplemental demands for surface water irrigation are met by Lake Okeechobee.
St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012 (see land use assumptions row). • Excess C-44 basin runoff is allowed to backflow into the Lake if lake stage is 0.25 ft. below the Zone D pulse release line before being pumped into the C-44 reservoir. • Basin demands include the Florida Power & Light reservoir at Indiantown. • Indian River Lagoon South Project features o Ten-mile Creek Reservoir and STA: 7,078 acre-feet storage capacity at 10.79 maximum depth on 820 acre footprint; receives excess water from North Folk Basin; o C-44 reservoir: 50,246 acre-feet storage capacity at 5.18 feet maximum depth on 12,125 acre footprint; C44 reservoir releases water back to Lake Okeechobee when Lake stages are below the bottom of the Baseflow Zone. o C-23/C-24 reservoir: 92,094 acre-feet storage capacity at 13.27 maximum depth on 8,675 acre footprint; o C-23/C-24 STA: 3,852 acre-feet storage capacity at 1.5 maximum depth on 2,568 acre footprint; and o All proposed reservoirs meet estuary demands.

Feature	
Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons per month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM. • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Everglades Agricultural Area	• Model water-body components as shown in Figure 1. • Simulated runoff from the North New River – Hillsboro basin apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 routes water from S-5A Basin to Hillsboro Basin. • RSMBN ECB EAA runoff and irrigation demand compared to SFWMM ECB simulated runoff and demand from 1965-2005 for reasonability.
Everglades Construction Project Stormwater Treatment Areas	• STAs are simulated as single waterbodies • STA-1E: 6,546 acres total area • STA-1W: 7,488 acres total area • S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E • STA-2: cells 1,2 & 3: 7,681 acres total area • STA-2N: cells 4,5 & 6; refers to Comp B-North; 6,531 acres total area • STA-2S: cells 7 & 8; refers to Comp B-South; 3,570 acres total area • STA-3/4: 17,126 acres total area • STA-5N: includes cells 1 & 2: 5,081 acres total area • STA-5S: includes cells 3, 4 & 5; uses footprint of Compartment C: 8,469 acres total area • STA-6: expanded with phase 2: 3,054 acres total area

Feature	
	• Assumed operations of STAs: o 0.5 ft minimum depth below which supply from external sources is triggered; o 4 ft maximum depth above which inflows are discontinued; and o Inflow targets established for STA-3/4, STA-2N and STA-2S based on DMSTA simulation; met from local basin runoff, LOK flood releases and available FEB storage. • A 29,617-acre Flow Equalization Basin (FEB) located north of STA-3/4 and Holeyland. The total footprint represents the original 15,853-acre A-1 footprint plus the additional 13,764-acre A-2 footprint operated as follows: o Assumed average topography of 9.63 ft NGVD. FEB inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south; o FEB outflows are used to help meet established inflow targets at STA-3/4, STA-2N, and STA-2S if EAA basin runoff and LOK flood releases are not sufficient; o 0.5 ft minimum depth below which no releases are allowed; o 3.8 ft maximum depth above which inflows are discontinued; o No supplemental water supply provided to FEB; o Assumed inlet pump from STA-3/4 supply canal with capacity equal to combined capacity of G-372 and G-370 structures; and o Outflow weirs, with similar discharge characteristics as STA-3/4 outlet structure, discharging into lower Miami and lower North New River canals.
Holey Land Wildlife Management Area	• G-372HL is the only inflow structure for Holey Land used for keeping the water table from going lower than half a foot below land surface elevation. • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FL Fish and Wildlife Conservation (FWC) Commission and the SFWMD.
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA. (SFWMD, March 2010)
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL FWO.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5N; G508 flows routed to STA5S; G406 flows routed to STA6. • C139 basin demand is met primarily by local groundwater.

Feature	
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan.



Water-Body Components:
Miami Water-Body = S3 + S8 + A-2WW
NNR/HILLS Water-Body = S2 + S6 + S7 + New Hope South
WPB Water-Body = S-5A
FEB = A-2W + A-2E + A-1

Fig. 1 RSMBSN Basin Definition within the EAA: Alternative 4 Revised (ALT4R) Scenario

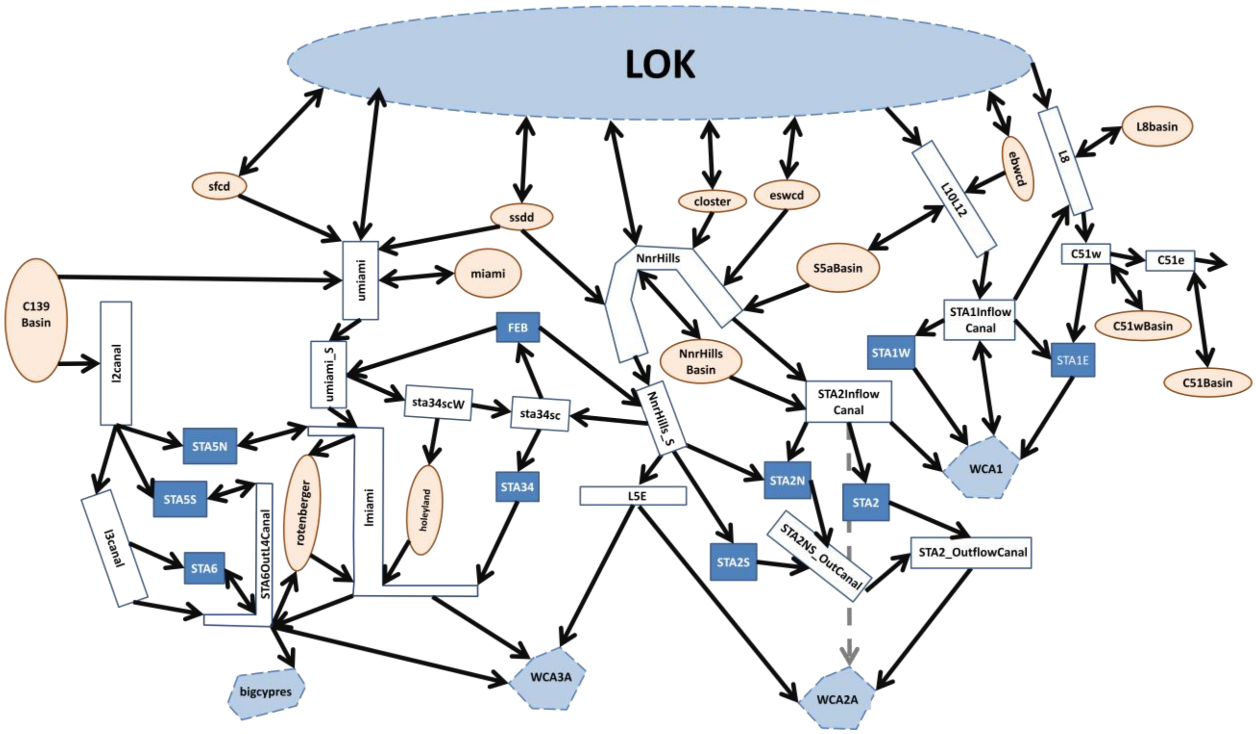


Fig. 2 RSMBSN Link-Node Routing Diagram: Alternative 4 Revised (ALT4R) Scenario

Note:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the eastern and southern boundaries of the RSMBN model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Glades-LECSA Model (RSMGL). The SFWMM was the source of the eastern boundary groundwater/surface water flows, while the RSMGL was the source of the southern boundary structural flows.

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Glades-LECSA (RSMGL)
Tentatively Selected Plan (ALT4R)
Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area.Modified at locations where reservoirs are introduced (STA1-E, Site 1 Impoundment, Broward WPAs, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed. • 8.5 SMA seepage canal is modeled as a WCD in ENP area.
Lake Belt Lakes	• Based on the permitted 2020 Lake Belt Lakes coverage obtained from USACE.
CERP Projects	• 1st Generation CERP – Site 1 Impoundment project is modeled as an above ground reservoir of area 1600 acres, with a maximum depth of 8 ft. • 2nd Generation CERP – Broward County Water Preserve Areas (WPAs) comprised of C-11 and C-9 impoundments were modeled as above ground reservoirs with areas 1221 and 1971 acres and maximum depths 4.3 and 4.0 ft. respectively. • 2nd Generation CERP – C-111 Spreader Canal Project includes the

Feature	
	Frog Pond Detention Area, which is modeled as an above ground impoundment with the S200 A, B and C pumps as inflow structures. In addition, the Aerojet canal is modeled with the inflow pumps S199 A, B and C. The S199 and S200 pumps are turned off based on the stage at the remote monitoring location EVER4 for the protection of the CSS Critical Habitat Unit 3. • 2nd Generation CERP – Biscayne Bay Coastal Wetlands project features were not modeled since these features along the coast in Miami-Dade County were not considered significant for CEPP. • Areal corrections were applied to the impoundment storages to account for the discrepancies of the areas in the model of the impoundments not matching the design areas.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels in WCA-2A are less than minimum operating criteria of 10.5 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Diversion of L-6 flows with additional 500 cfs structure and improvements to the L-5 canal • STA-3/4 outflows routed based on Rainfall Driven Operations (RDO) – a maximum of 2500 cfs is routed to S8 and G404, with the remainder being sent to S7 • Western L-4 levee degrade (west of S-8 = 3,000 cfs capacity) • Miami Canal backfilled and spoil mound removed 1.5 miles south of S-8 to I-75 • L-28 Triangle – levee gap and canal backfill approximately 9000 ft. • Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 (USACE, 2012) • One 500 cfs gated structure in L-67A north of Blue Shanty levee (S345D) and associated gap in L-67C levee • Two 500 cfs gated structures in L-67A (S345F & S345G) discharging into Blue Shanty Flowway

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<table>
<tr><th>Feature</th><th></th></tr>
<tr><td></td><td>Environmental target deliveries through the S345s are determined through RDO and is spatially distributed as 40% to 345D, 35% to 345F and 25% to 345G
Blue Shanty Flowway assumed as follows:
Construction of ~8.5 mile levee in WCA 3B, connecting L-67A to L-29
Removal of L-67C levee in Blue Shanty Flowway (no canal back fill)
Removal of L-29 levee in Blue Shanty Flowway.
Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2002)
No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A, defined as when 3-69W marsh gauge falls below 7.5 ft or CA3 canal stage fall below 7.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.</td></tr>
<tr><td>Everglades Construction Project Stormwater Treatment Areas</td><td>STA-1E: 5,132 acres total treatment area.
A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.</td></tr>
<tr><td>Everglades National Park</td><td>Water deliveries to Everglades National Park are based upon Everglades Restoration Transition Plan (ERTP), with the WCA-3A Regulation Schedule including the lowered Zone A (compared to IOP) and extended Zones D and E1. The environmental component of the schedule is defined by RDO. If hydraulic capacity exists at the 345s, then flood control discharges are made into 3B instead of at the S12s.
S-333 capacity increased to 2,500 cfs
L29 Divide structure assumed and is operated to send water from L29W to L29E to equilibrate canals when L29E falls below 7 ft.
L29 canal can receive inflow up to 9.7 ft (applies to both E and W segments / i.e. S333 & S356)
G-3273 constraint for operation of S-333 assumed to be 9.5 ft, NGVD.
The one mile Tamiami Trail Bridge as per the 2008 Tamiami Trail Limited Reevaluation Report is modeled as a one mile weir. Located east of the L67 extension and west of the S334 structure.
Western 2.6 mile Tamiami Trail Bridge, modeled as a 2.6 mile long weir, and is located east of Osceola Camp and west of Frog City.
Tamiami Trail culverts east of the L67 Extension are simulated where the bridge is not located.
Removal of the entire 5.5 miles L-67 Extension levee, with backfill of L-67 Extension canal
S-355A & S-355B are operated.</td></tr>
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Feature	
	• Capacity of S-356 pump increased to 1000 cfs. S-356 is operated to manage seepage. • Full construction of C-111 project reservoirs consistent with the as-built information from USACE plus addition of contract 8 and contract 9 features. A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed. • 8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2. Outflow assumed from 8.5 SMA detention cell to the C-111 North Detention Area. ○ An additional length of seepage canal is assumed in the model to allow water to be collected for S357 operation. • Partial depth, approximately 4 mile long seepage barrier south of Tamiami Trail (along L-31N)
Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay
Pumpage and Irrigation	• Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included • Residential Self Supported (RSS) pumpage are based on 2030 projections of residential population from the SFWMD Water Supply Bureau. • Industrial pumpage is also based on 2030 projections of industrial use from the Water Supply Bureau. • Irrigation demands for the six irrigation land-use types are calculated internally by the model. • Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.
Canal Operations	• C&SF system and operating rules in effect in 2012 • Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion • Includes existing secondary drainage/water supply system • C-4 Flood Mitigation Project • Western C-4, S-380 structure retained open • C-11 Water Quality Treatment Critical Project (S-381 and S-9A) • S-25B and S-26 backflow pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary • Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry • ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure

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Feature	
	• Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 o Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15 o Structure S-12B is closed Jan. 1 to July 15 • Water supply deliveries from regional system (from WCA3A: S-151/S-337) are used to maintain the L30 canal with a minimum seasonal level varying from 6.25 ft in the dry season to 5.2 ft. at the beginning of the wet season • G-211 / S338 operational refinements; use coastal canals to convey seepage toward Biscayne Bay during drier times.
Canal Configuration	• Canal configuration same as calibration except no L-67 Extension Canal and CERP & CEPP project modifications.
Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data. • Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the corresponding RSMBN FWO simulation.

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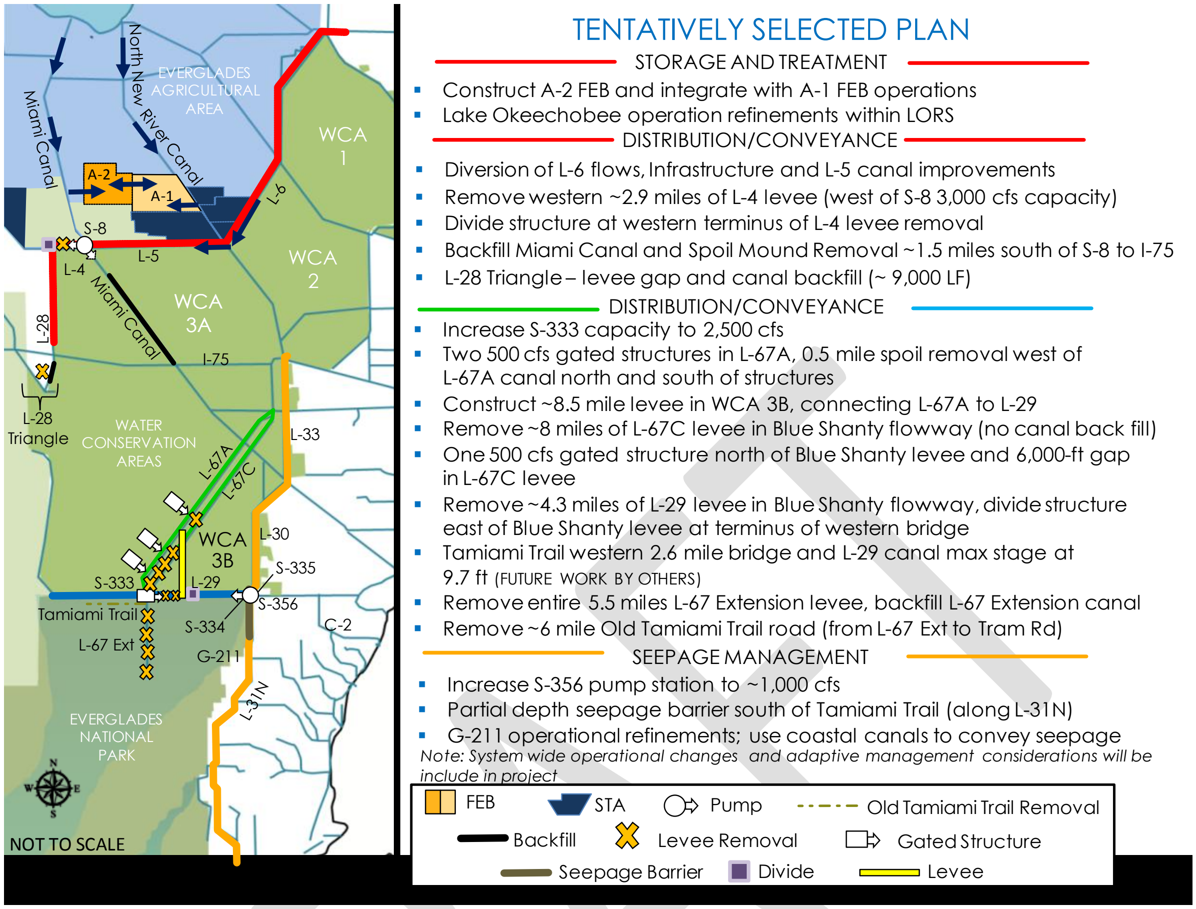

Fig. 1 CEPP ALT4R Features as defined by CEPP project team

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.

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Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Basins (RSMBN) Updated Tentatively Selected Plan (ALT4R2) Table of Assumptions

Feature	
Climate	• The climatic period of record is from 1965 to 2005. • Rainfall estimates have been revised and updated for 1965-2005. • Revised evapotranspiration methods have been used for 1965-2005.
Topography	The Topography dataset for RSM was Updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004; • High Accuracy Elevation Data, US Geological Survey 2007; • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004; • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007; • Palm Beach County LiDAR Surve, Dewberry and Davis, 2004; and • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/2012, as reflected in the LOSA Ledger produced by the Water Use Bureau. • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information . • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass.
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).

Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008)
	  o CEPP optimized release guidance in order to improve selected performance within LOK, the northern estuaries and LOSA while meeting environmental targets in the Glades.
	  o Lake Okeechobee can send flood releases south through the Miami Canal and North New River Canal to the FEB when the LOK stage is above the bottom of Zone D and the FEB depth is below 2’ (EAA basin runoff used to limit conveyance capacity: 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal).
	  o Lake Okeechobee can send flood releases south to help meet water-quality based flow targets at STA-3/4, STA-2N, and STA-2S when the LOK stage is above the bottom of the Baseflow Zone (EAA basin runoff used to limit conveyance capacity: 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal).
	  o Includes Lake Okeechobee regulatory releases to tide via L8 canal.

Feature	
	o Releases via S-77 can be diverted into C43 Reservoir • Lake Okeechobee Water Shortage Management (LOWSM) Plan. • Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and S-3 is to be minimized). • "Temporary" forward pumps as follows: o S354 – 400 cfs o S351 – 600 cfs o S352 – 400 cfs o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages recover to greater than 11.2 ft • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Headwaters Revitalization schedule for Kissimmee Chain of Lakes using the UKISS model. • Kissimmee River Restoration complete. • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012. (see land use assumptions row) • Public water supply daily intake from the river is included in the analysis. • Maximum reservoir height of 41.7 ft NGVD with a 9,379-acre footprint in Western C43 basin with a 175,800 acre-feet effective storage. • Proposed reservoir meets estuary demands while C-43 basin supplemental demands for surface water irrigation are met by Lake Okeechobee.
St. Lucie Canal Basin	o St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012 (see land use assumptions row). o Excess C-44 basin runoff is allowed to backflow into the Lake if lake stage is 0.25 ft. below the Zone D pulse release line before being pumped into the C-44 reservoir. o Basin demands include the Florida Power & Light reservoir at Indiantown. o Indian River Lagoon South Project features o Ten-mile Creek Reservoir and STA: 7,078 acre-feet storage capacity at 10.79 maximum depth on 820 acre footprint; receives excess water from North Folk Basin; o C-44 reservoir: 50,246 acre-feet storage capacity at 5.18 feet maximum depth on 12,125 acre footprint; C44 reservoir releases water back to Lake Okeechobee when Lake stages are below the bottom of the Baseflow Zone. o C-23/C-24 reservoir: 92,094 acre-feet storage capacity at 13.27 maximum depth on 8,675 acre footprint; o C-23/C-24 STA: 3,852 acre-feet storage capacity at 1.5 maximum depth on 2,568 acre footprint;

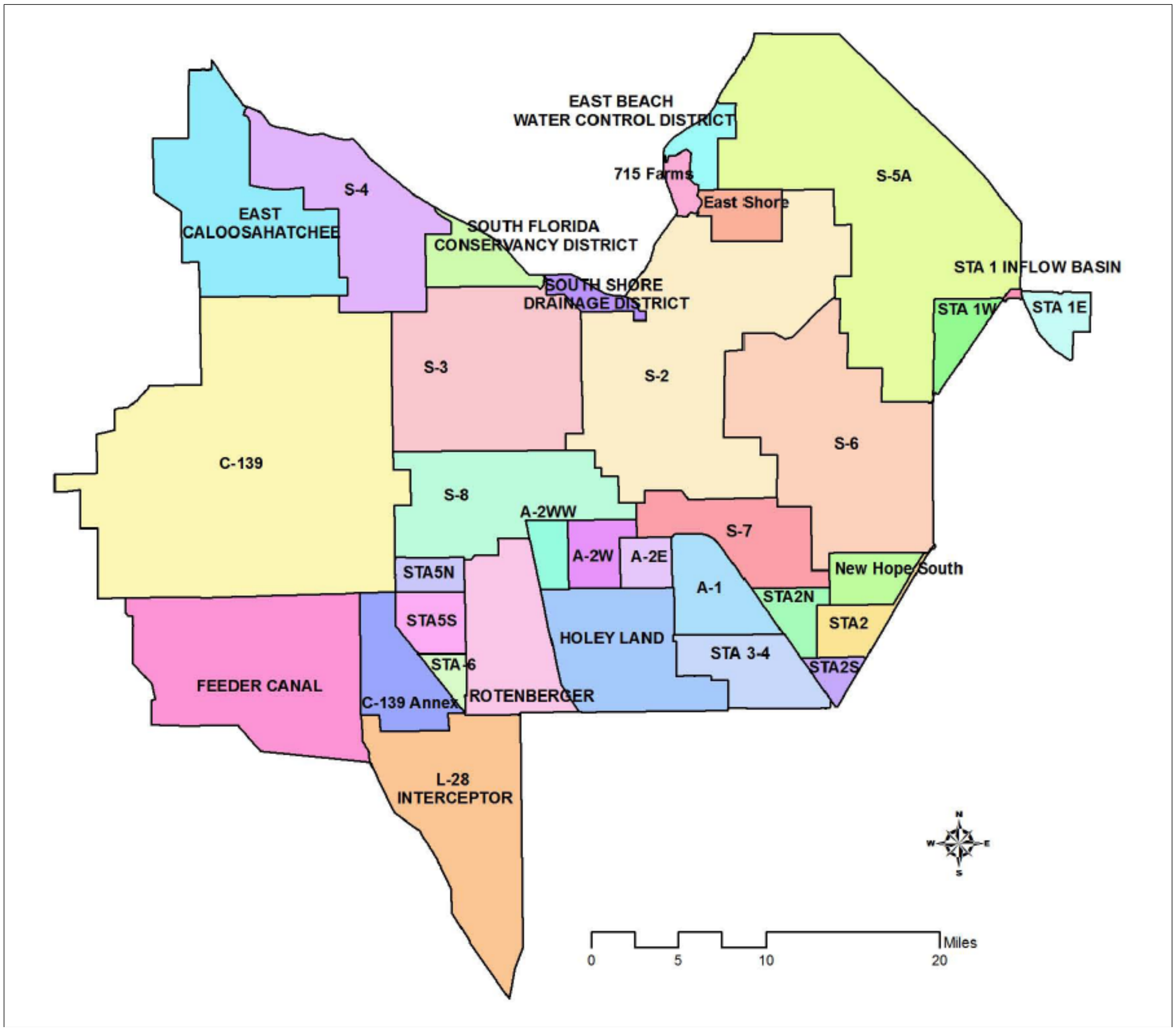
Feature	
	o All proposed reservoirs meet estuary demands. o IRL operations assumed are consistent with the March 2010 St. Lucie River Water Reservation Rule update. o Excess C23 basin water not needed to meet estuary demands can be diverted to the C44 reservoir if capacity exists. o C44 reservoir can discharge to C44 canal and backflow to Lake Okeechobee when the lake is below the baseflow zone.
Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons per month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM. • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Everglades Agricultural Area	• Model water-body components as shown in Figure 1. • Simulated runoff from the North New River – Hillsboro basin apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 routes water from S-5A Basin to Hillsboro Basin. • RSMBN ECB EAA runoff and irrigation demand compared to SFWMM ECB simulated runoff and demand from 1965-2005 for reasonability.
Everglades Construction Project Stormwater Treatment Areas	• STAs are simulated as single waterbodies • STA-1E: 6,546 acres total area • STA-1W: 7,488 acres total area • S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E • STA-2: cells 1,2 & 3: 7,681 acres total area • STA-2N: cells 4,5 & 6; refers to Comp B-North; 6,531 acres total area • STA-2S: cells 7 & 8; refers to Comp B-South; 3,570 acres total area • STA-3/4: 17,126 acres total area

Feature	
	• STA-5N: includes cells 1 & 2: 5,081 acres total area • STA-5S: includes cells 3, 4 & 5; uses footprint of Compartment C: 8,469 acres total area • STA-6: expanded with phase 2: 3,054 acres total area • Assumed operations of STAs: o 0.5 ft minimum depth below which supply from external sources is triggered; o 4 ft maximum depth above which inflows are discontinued; and o Inflow targets established for STA-3/4, STA-2N and STA-2S based on DMSTA simulation; met from local basin runoff, LOK flood releases and available FEB storage. • A 29,617-acre Flow Equalization Basin (FEB) is located north of STA-3/4 and Holeyland. The total footprint represents the original 15,853-acre A-1 footprint plus the additional 13,764-acre A-2 footprint operated as follows: o Assumed average topography of 9.63 ft NGVD. FEB inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south; o FEB outflows are used to help meet established inflow targets at STA-3/4, STA-2N, and STA-2S if EAA basin runoff and LOK flood releases are not sufficient; o 0.5 ft minimum depth below which no releases are allowed; o 3.8 ft maximum depth above which inflows are discontinued; o No supplemental water supply provided to FEB; o Assumed inlet pump from STA-3/4 supply canal with capacity equal to combined capacity of G-372 and G-370 structures; and o Outflow weirs, with similar discharge characteristics as STA-3/4 outlet structure, discharging into lower Miami and lower North New River canals.
Holey Land Wildlife Management Area	• G-372HL is the only inflow structure for Holey Land used for keeping the water table from going lower than half a foot below land surface elevation. • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FL Fish and Wildlife Conservation (FWC) Commission and the SFWMD.
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA. (SFWMD, March 2010)
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL FWO.

Feature	
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5N; G508 flows routed to STA5S; G406 flows routed to STA6. • C139 basin demand is met primarily by local groundwater.
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan.

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the eastern and southern boundaries of the RSMBN model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Glades-LECSA Model (RSMGL). The SFWMM was the source of the eastern boundary groundwater/surface water flows, while the RSMGL was the source of the southern boundary structural flows.
- The RSMBN CEPP representation of ALT4R2 is the same as the June 2, 2013 ALT4R1 scenario.



Water-Body Components:
Miami Water-Body = S3 + S8 + A-2WW
NNR/HILLS Water-Body = S2 + S6 + S7 + New Hope South
WPB Water-Body = S-5A
FEB = A-2W + A-2E + A-1

Fig. 1 RSMBSN Basin Definition within the EAA: Updated Tentatively Selected Plan (ALT4R2)

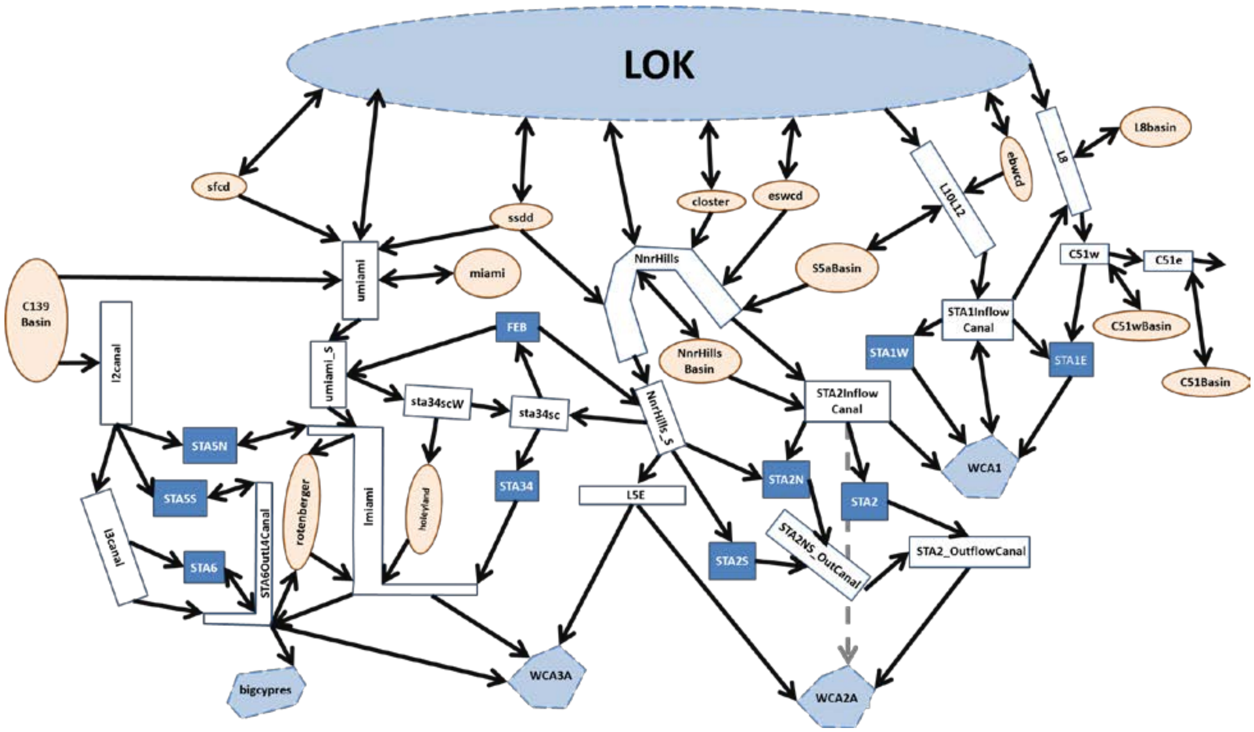


Fig. 2 RSMBSN Link-Node Routing Diagram: Updated Tentatively Selected Plan (ALT4R2)

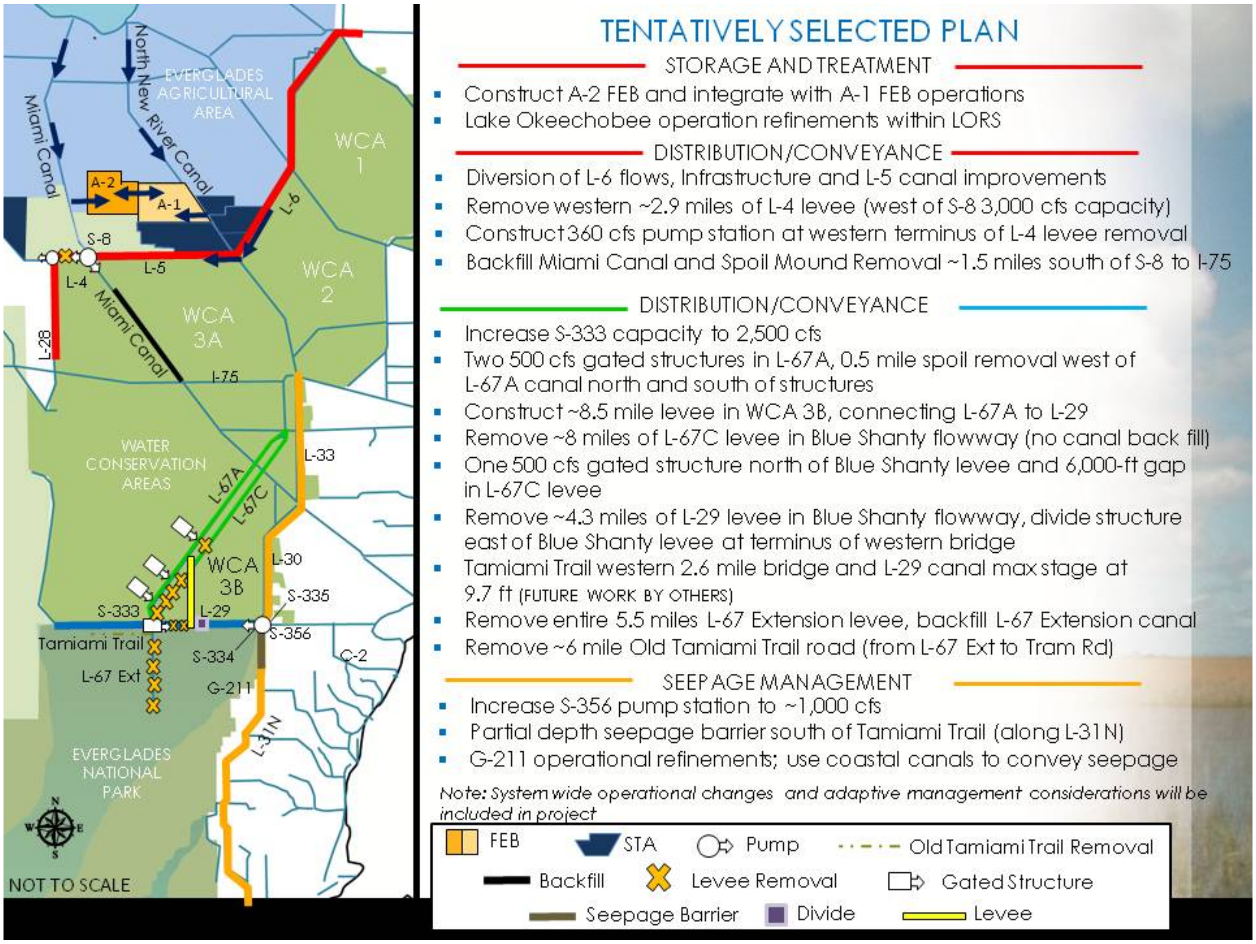


Fig. 3 CEPP ALT4R2 Features as defined by CEPP project team

Hydrologic and Environmental Systems Modeling & Interagency Modeling Center

Regional Simulation Model Glades-LECSA (RSMGL) Updated Tentatively Selected Plan (ALT4R2) Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area. Modified at locations where reservoirs are introduced (STA1-E, Site 1 Impoundment, Broward WPAs, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed. • 8.5 SMA seepage canal is modeled as a WCD in ENP area.
Lake Belt Lakes	• Based on the permitted 2020 Lake Belt Lakes coverage obtained from USACE.
CERP Projects	• 1st Generation CERP – Site 1 Impoundment project is modeled as an above ground reservoir of area 1600 acres, with a maximum depth of 8 ft. • 2nd Generation CERP – Broward County Water Preserve Areas (WPAs) comprised of C-11 and C-9 impoundments were modeled as above ground reservoirs with areas 1221 and 1971 acres and maximum depths 4.3 and 4.0 ft. respectively. Operations refined in RSM model to closer represent project intent and outcomes.

Feature	
	• 2nd Generation CERP – C-111 Spreader Canal Project includes the Frog Pond Detention Area, which is modeled as an above ground impoundment with the S200 A, B and C pumps as inflow structures. In addition, the Aerojet canal is modeled with the inflow pumps S199 A, B and C. The S199 and S200 pumps are turned off based on the stage at the remote monitoring location EVER4 for the protection of the CSS Critical Habitat Unit 3. • 2nd Generation CERP – Biscayne Bay Coastal Wetlands project features were not modeled since these features along the coast in Miami-Dade County were not considered significant for CEPP. • Areal corrections were applied to the impoundment storages to account for the discrepancies of the areas in the model of the impoundments not matching the design areas.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 10.5 ft in WCA-2A, defined as when WCA2-U1 marsh gauge falls below 10.5 ft or L38 canal stage falls below 10.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Diversion of L-6 flows with additional 500 cfs structure and improvements to the L-5 canal • STA-3/4 outflows routed based on Rainfall Driven Operations (RDO) – a maximum of 2500 cfs is routed to S8 and G404, with the remainder being sent to S7 • Western L-4 levee degrade with 1.5 miles retained west of S8 (west of S-8 = 3,000 cfs capacity) • Miami Canal backfilled and spoil mound removed 1.5 miles south of S-8 to I-75 • Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 (USACE, 2012) • One 500 cfs gated structure in L-67A north of Blue Shanty levee (S345D) and associated gap in L-67C levee

Feature	
	• Two 500 cfs gated structures in L-67A (S345F & S345G) discharging into Blue Shanty Flowway • Environmental target deliveries through the S345s are determined through RDO and is spatially distributed as 40% to 345D, 35% to 345F and 25% to 345G • Blue Shanty Flowway assumed as follows: o Construction of ~8.5 mile levee in WCA 3B, connecting L-67A to L-29 o Removal of L-67C levee in Blue Shanty Flowway (no canal back fill) o Removal of L-29 levee in Blue Shanty Flowway. • Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2002) • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A, defined as when 3-69W marsh gauge falls below 7.5 ft or CA3 canal stage falls below 7.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Everglades Construction Project Stormwater Treatment Areas	• STA-1E: 5,132 acres total treatment area. • A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.
Everglades National Park	• Water deliveries to Everglades National Park are based upon Everglades Restoration Transition Plan (ERTP), with the WCA-3A Regulation Schedule including the lowered Zone A (compared to IOP) and extended Zones D and E1. The environmental component of the schedule is defined by RDO. If hydraulic capacity exists at the 345s, then flood control discharges are made into 3B instead of at the S12s. • S-333 capacity increased to 2,500 cfs • L29 Divide structure assumed and is operated to send water from L29W to L29E to equilibrate canals when L29E falls below 7 ft. • L29 canal can receive inflow up to 9.7 ft (applies to both E and W segments / i.e. S333 & S356 as well as S345F & S345G structure on Blue Shanty Flowway) • G-3273 constraint for operation of S-333 assumed to be 9.5 ft, NGVD. • The one mile Tamiami Trail Bridge as per the 2008 Tamiami Trail Limited Reevaluation Report is modeled as a one mile weir. Located east of the L67 extension and west of the S334 structure. • Western 2.6 mile Tamiami Trail Bridge, modeled as a 2.6 mile long weir, and is located east of Osceola Camp and west of Frog City. • Tamiami Trail culverts east of the L67 Extension are simulated where the bridge is not located.

Feature	
	• Removal of the entire 5.5 miles L-67 Extension levee, with backfill of L-67 Extension canal • S-355A & S-355B are operated. • Capacity of S-356 pump increased to 1000 cfs. S-356 is operated to manage seepage. • Full construction of C-111 project reservoirs consistent with the as-built information from USACE plus addition of contract 8 and contract 9 features. A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed. • 8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2. Outflow assumed from 8.5 SMA detention cell to the C-111 North Detention Area. o An additional length of seepage canal is assumed in the model to allow water to be collected for S357 operation. • Partial depth, approximately 4 mile long seepage barrier south of Tamiami Trail (along L-31N)
Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay
Pumpage and Irrigation	• Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included • Modeling of the TSP assumes an additional public water supply withdrawal of 12 MGD in Service Area 2 and 5 MGD in Service Area 3. • Residential Self Supported (RSS) pumpage are based on 2030 projections of residential population from the SFWMD Water Supply Bureau. • Industrial pumpage is also based on 2030 projections of industrial use from the Water Supply Bureau. • Irrigation demands for the six irrigation land-use types are calculated internally by the model. • Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.
Canal Operations	• C&SF system and operating rules in effect in 2012 • Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion • Includes existing secondary drainage/water supply system • C-4 Flood Mitigation Project • Western C-4, S-380 structure retained open • C-11 Water Quality Treatment Critical Project (S-381 and S-9A) • S-25B and S-26 backflow pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary

Feature	
	• Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry • ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure • Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 o Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15 o Structure S-12B is closed Jan. 1 to July 15 • Water supply deliveries from regional system (from WCA3A: S-151/S-337) are used to maintain the L30 canal with a minimum seasonal level varying from 6.25 ft in the dry season to 5.2 ft. at the beginning of the wet season • G-211 / S338 operational refinements; use coastal canals to convey seepage toward Biscayne Bay during drier times.
Canal Configuration	• Canal configuration same as calibration except no L-67 Extension Canal and CERP & CEPP project modifications.
Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data. • Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the corresponding RSMBN FWO simulation.

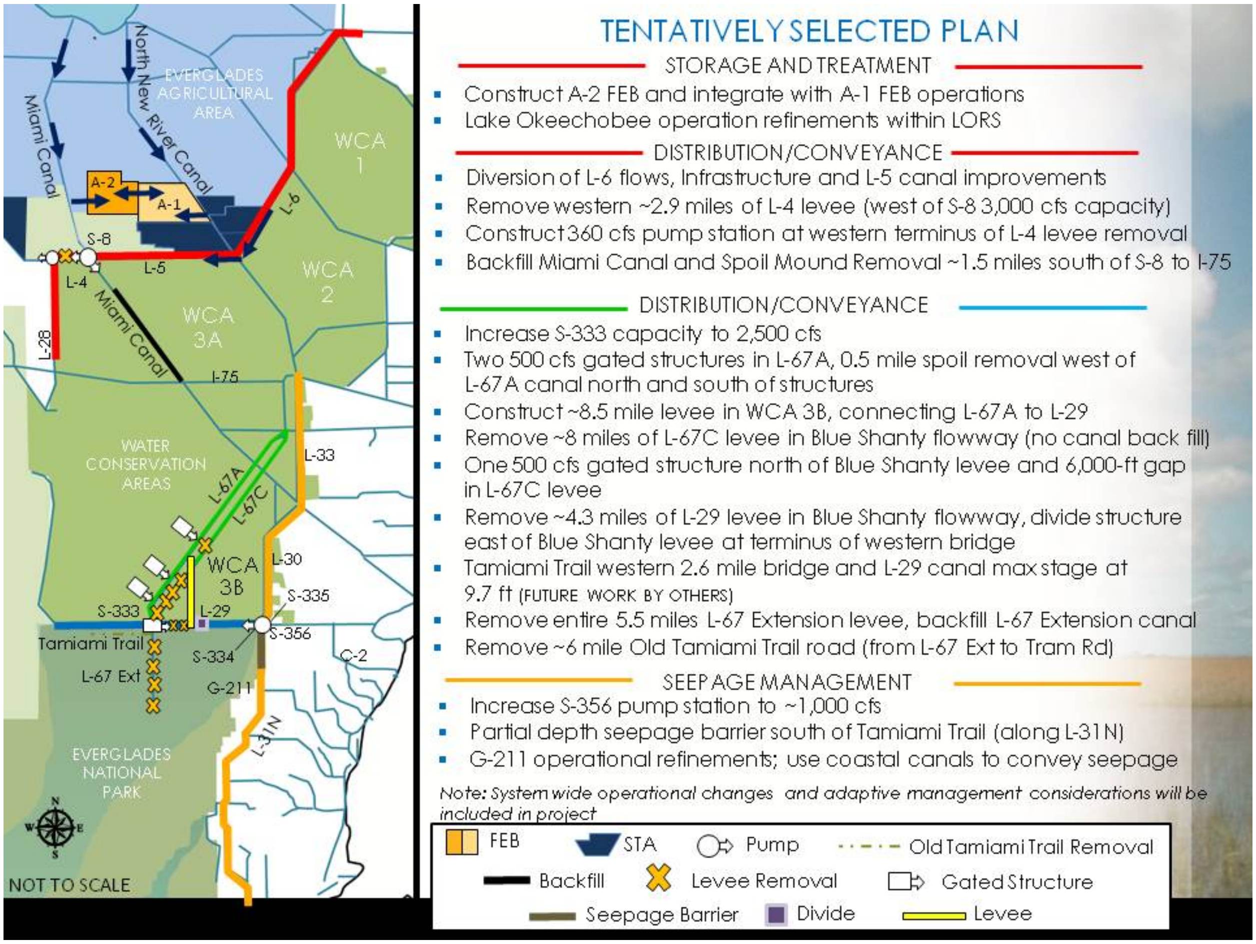


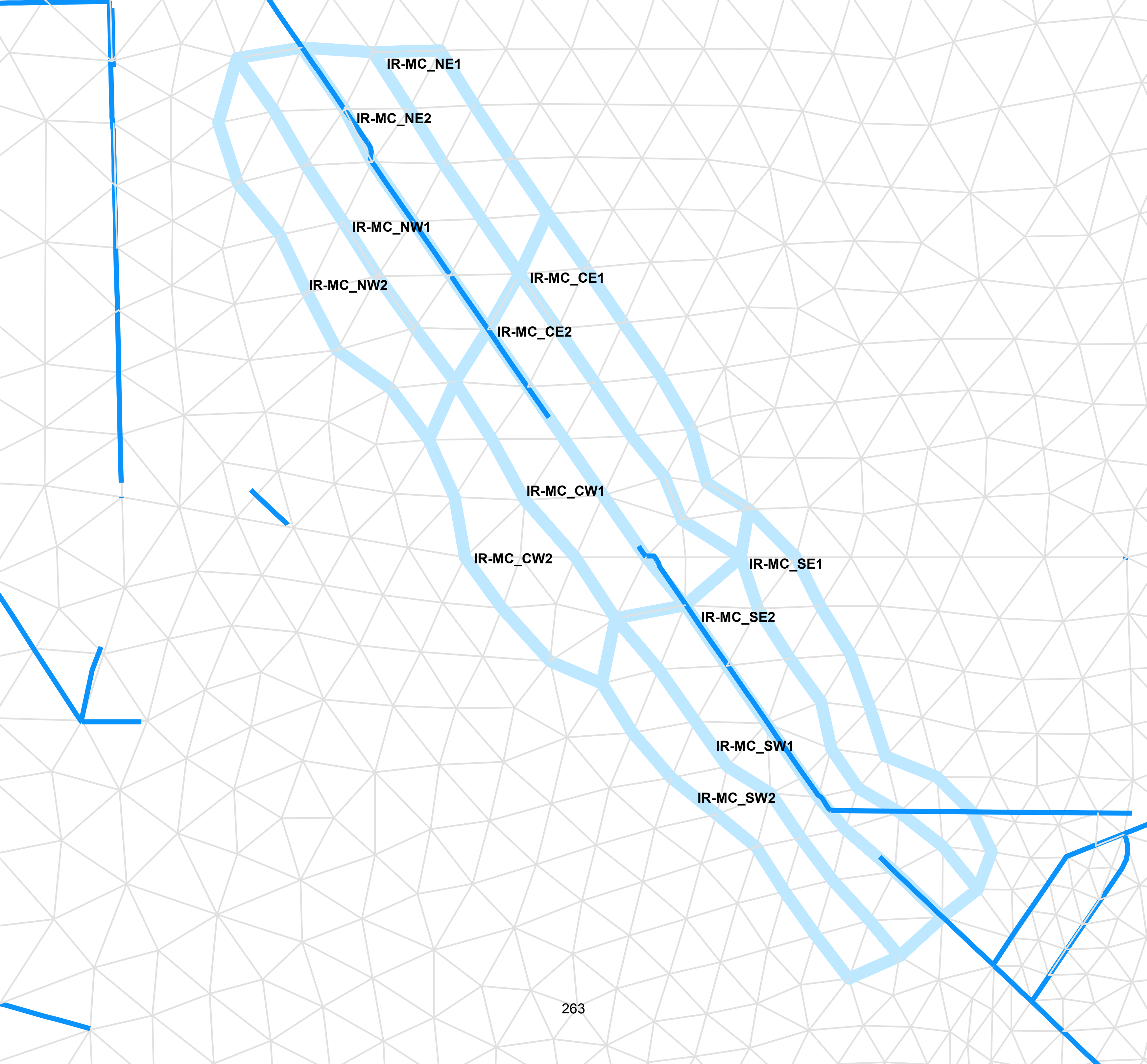
Fig. 1 CEPP ALT4R2 Features as defined by CEPP project team

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.

ANNEX A-2: REFERENCE 3

RSM-GL REFERENCE MAPS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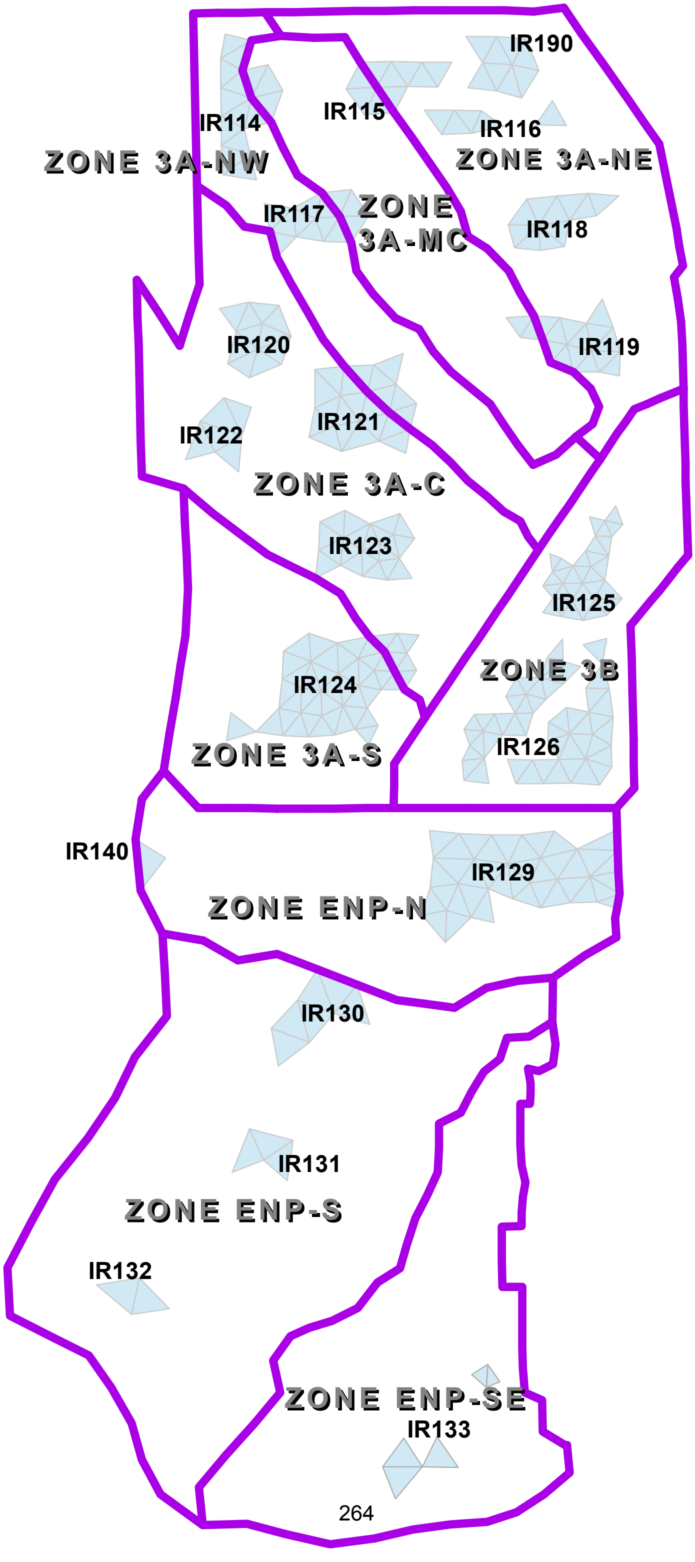
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IR116
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IR117
ZONE 3A-MC
IR118
IR120
IR119
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IR122
ZONE 3A-C
IR123
IR125
ZONE 3B
IR124
IR126
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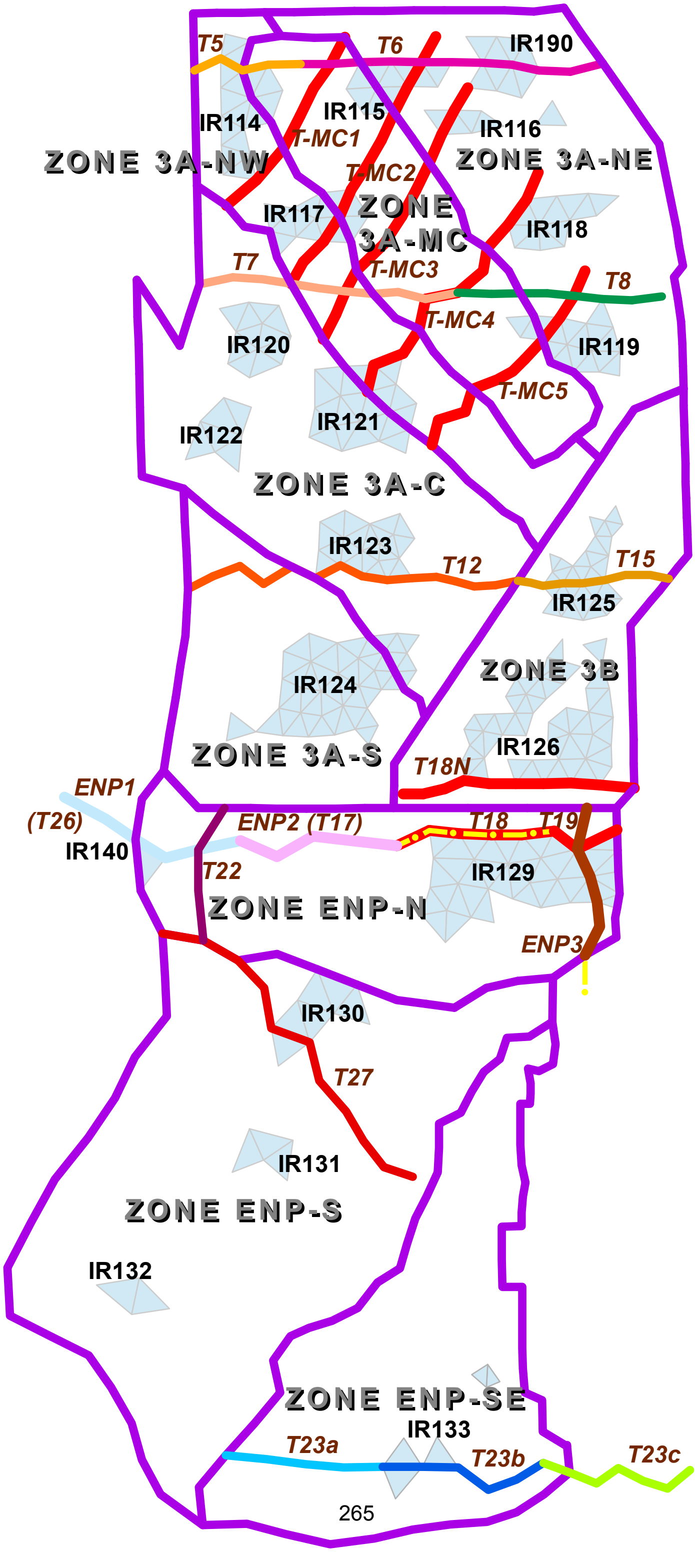
ZONE ENP-S
IR132
ZONE ENP-SE
IR133



ZONE 3A-NW
ZONE 3A-NE
ZONE 3A-MC
ZONE 3A-C
ZONE 3A-S
ZONE 3B
ZONE ENP-N
ZONE ENP-S
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T8
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T-MC2
T-MC3
T-MC4
T-MC5
T12
T15
T18N
T18
T19
T22
T23a
T23b
T23c
T27
ENP1
(T26)
ENP2 (T17)
ENP3
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IR114
IR115
IR116
IR117
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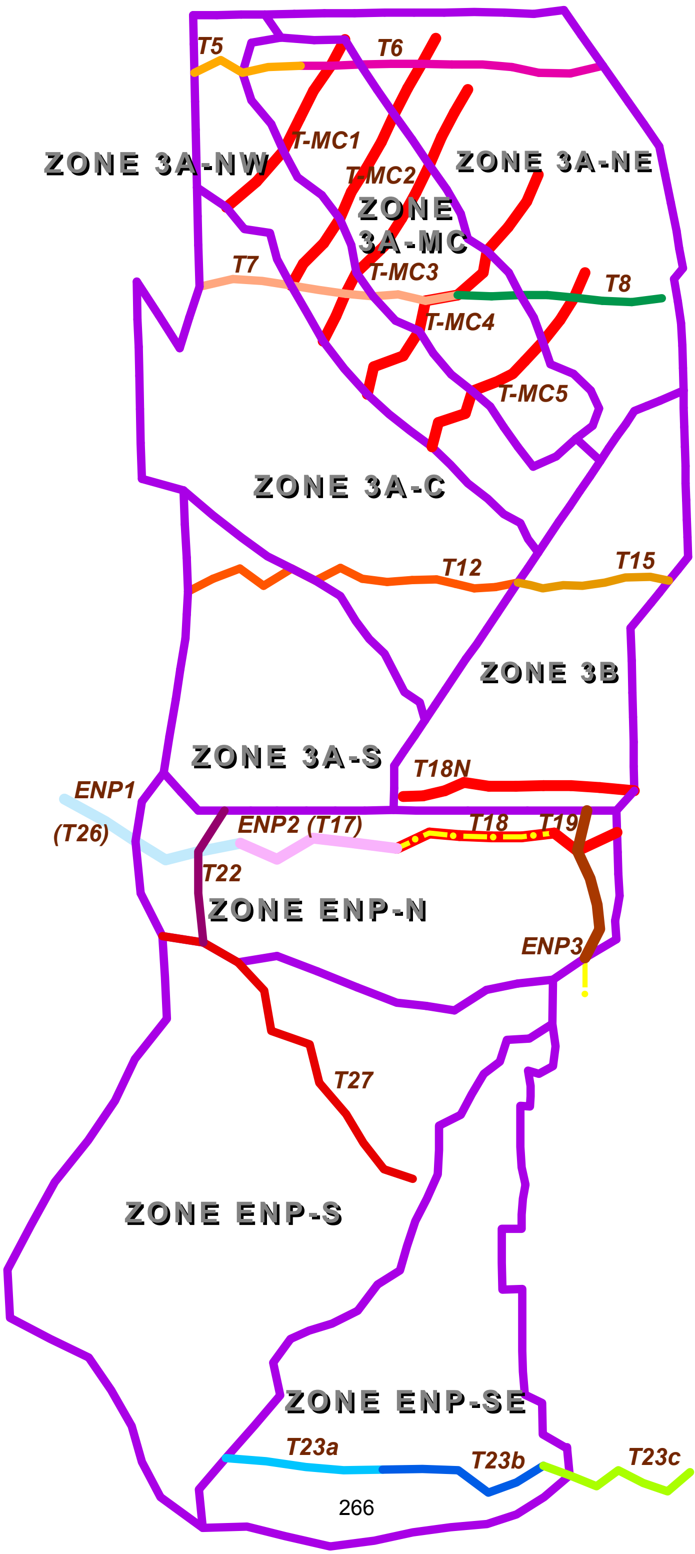
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T12
T15
ZONE 3B
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(T26)
ENP2 (T17)
T18
T19
T22
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T23a
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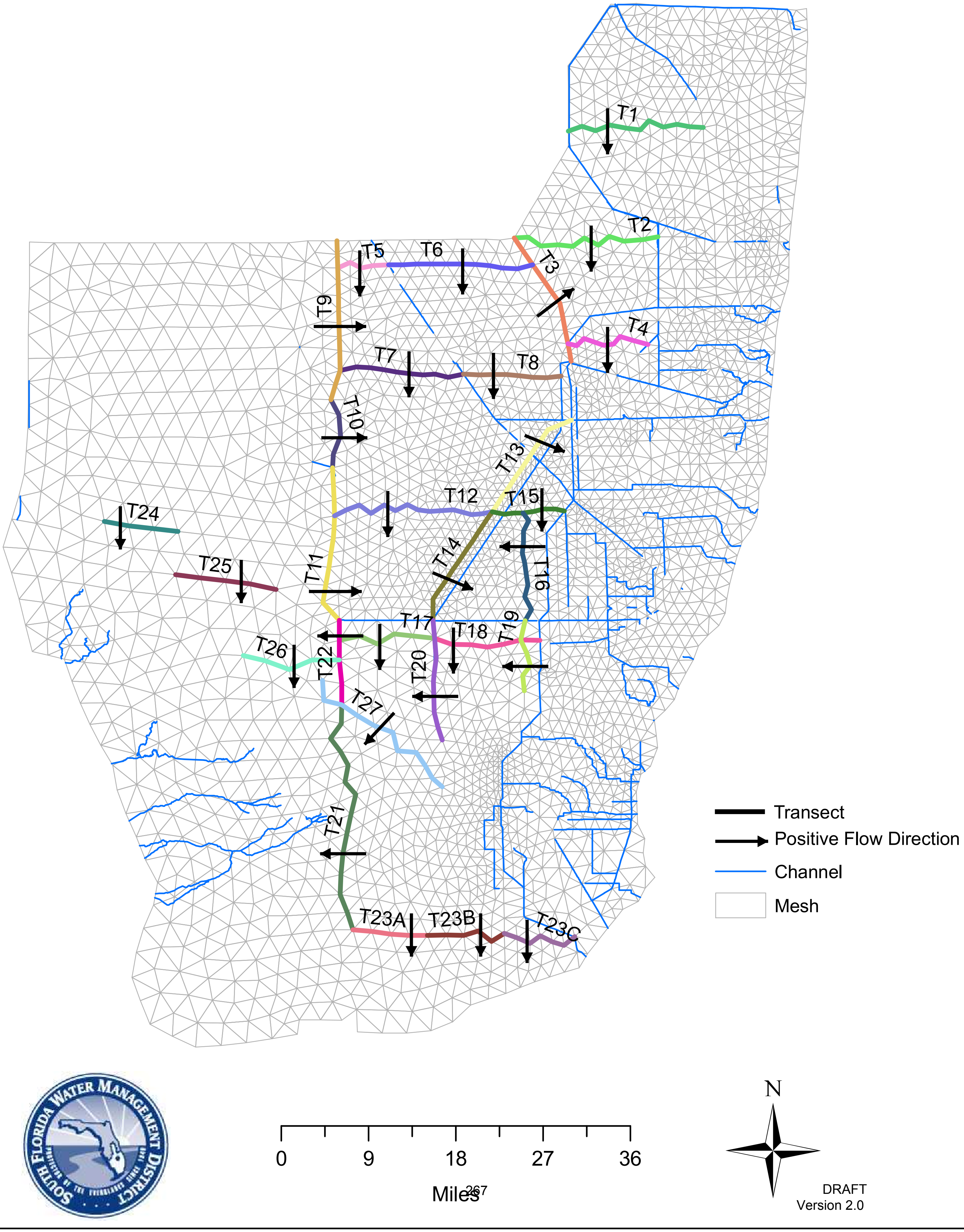
RSM Glades-LECSA - Transects



DRAFT
Version 2.0

Transects used to report average annual overland flow

Transect	Description
T1	Southward flow in WCA-1
T2	Southward flow in WCA-2A
T3	Flow across L-38 (from WCA-3A to WCA-2)
T4	Southward flow in WCA-2B
T5	Southward flows in Northern WCA-3A (west of Miami Canal)
T6	Southward flows in Northern WCA-3A (east of Miami Canal)
T7	Southward flow in Central WCA-3A (south of Alligator Alley & west of Miami Canal)
T8	Southward flow in Central WCA-3A (south of Alligator Alley & east of Miami Canal)
T9	Eastward flows across North Western WCA-3A boundary
T10	Eastward flows across Central WCA-3A boundary
T11	Eastward flows across South Western WCA-3A boundary
T12	Southward flow in Southern WCA-3A
T13	Flows across L-67 North (from WCA-3A to WCA-3B)
T14	Flows across L-67 South (from WCA-3A to WCA-3B)
T15	Southward flow in Northern WCA-3B
T16	Westward flow in Eastern WCA-3B
T17	Southward flows in Northern ENP (South of Tamiami Trail & West of L-67 extension)
T18	Southward flows in Northern ENP (South of Tamiami Trail & East of L-67 extension)
T19	Westward flow in North Eastern ENP (west of L-31N & north of G-211)
T20	Westward flow in North-Central ENP (south of Tamiami Trail at L-67 extension)
T21	Westward flow in Western Shark River Slough
T22	Westward flow in North Western Shark River Slough
T23A	Southward flow in Southern ENP (Craighead Basin)
T23B	Southward flow in Southern ENP (Taylor Slough)
T23C	Southward flow in Southern ENP (Eastern Panhandle)
T24	Southward flow in South-Western BCNP
T25	Southward flow in South-Eastern BCNP
T26	Southward flow in Lostmans
T27	Southwestward flow in Central Shark River Slough

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

FOR INTERNAL SOUTH FLORIDA WATER MANAGEMENT DISTRICT USE ONLY

RSM Glades - LECSA
CEPP - Levee/Canal Segments

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

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RSM Glades - LECSA
CEPP - Gauge/Cell Locations

For copies of this map (Z:\current_storm\maps\base_maps\EOC_Basemap.mxd)which was produced on 6/01/2011 by GIS EOC Staff, contact the GIS Section

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

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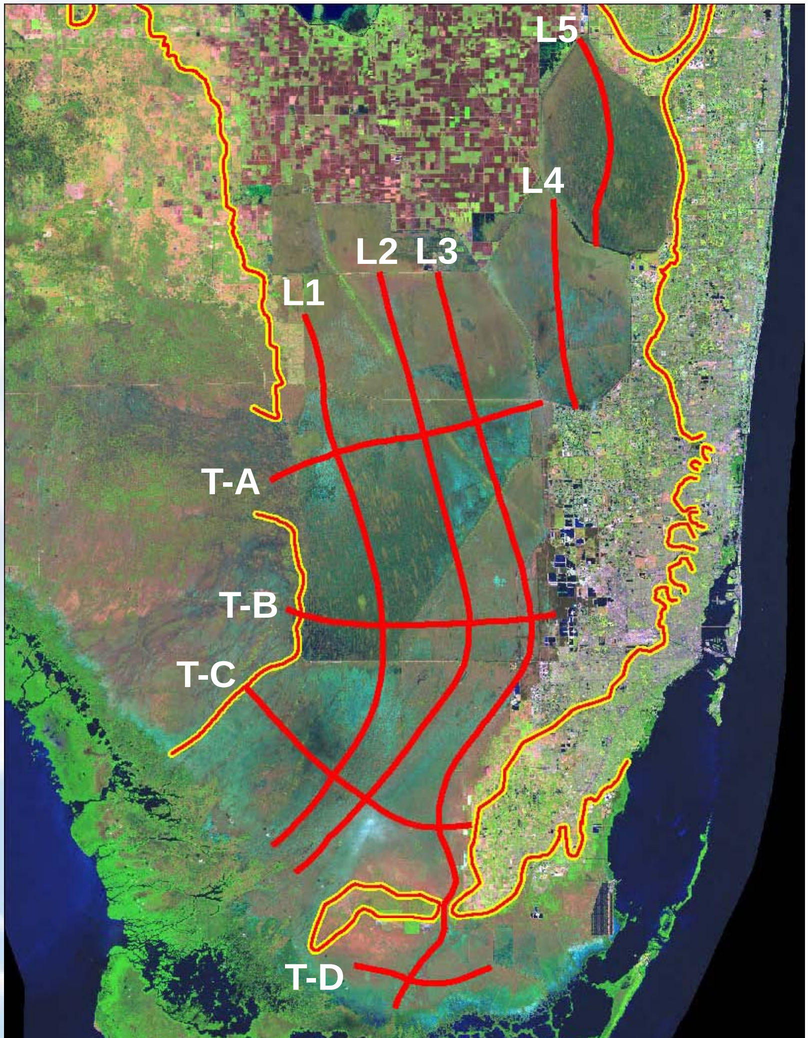
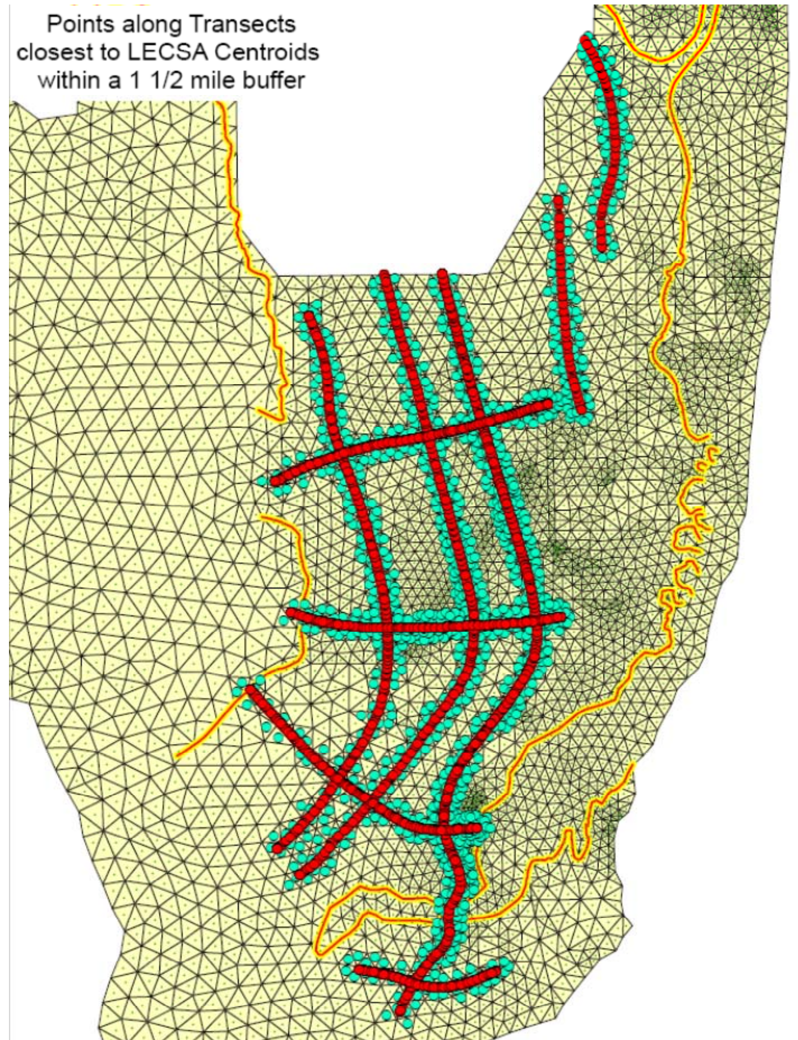
RSM Glades - LECSA
Hydrological Area Near Camps

For copies of this map (Z:\current_storm\maps\base_maps\EOC_Basemap.mxd)which was produced on 6/01/2011 by GIS EOC Staff, contact the GIS Section

Aligned with Landscape Directionality

RESTORATION PLANNING





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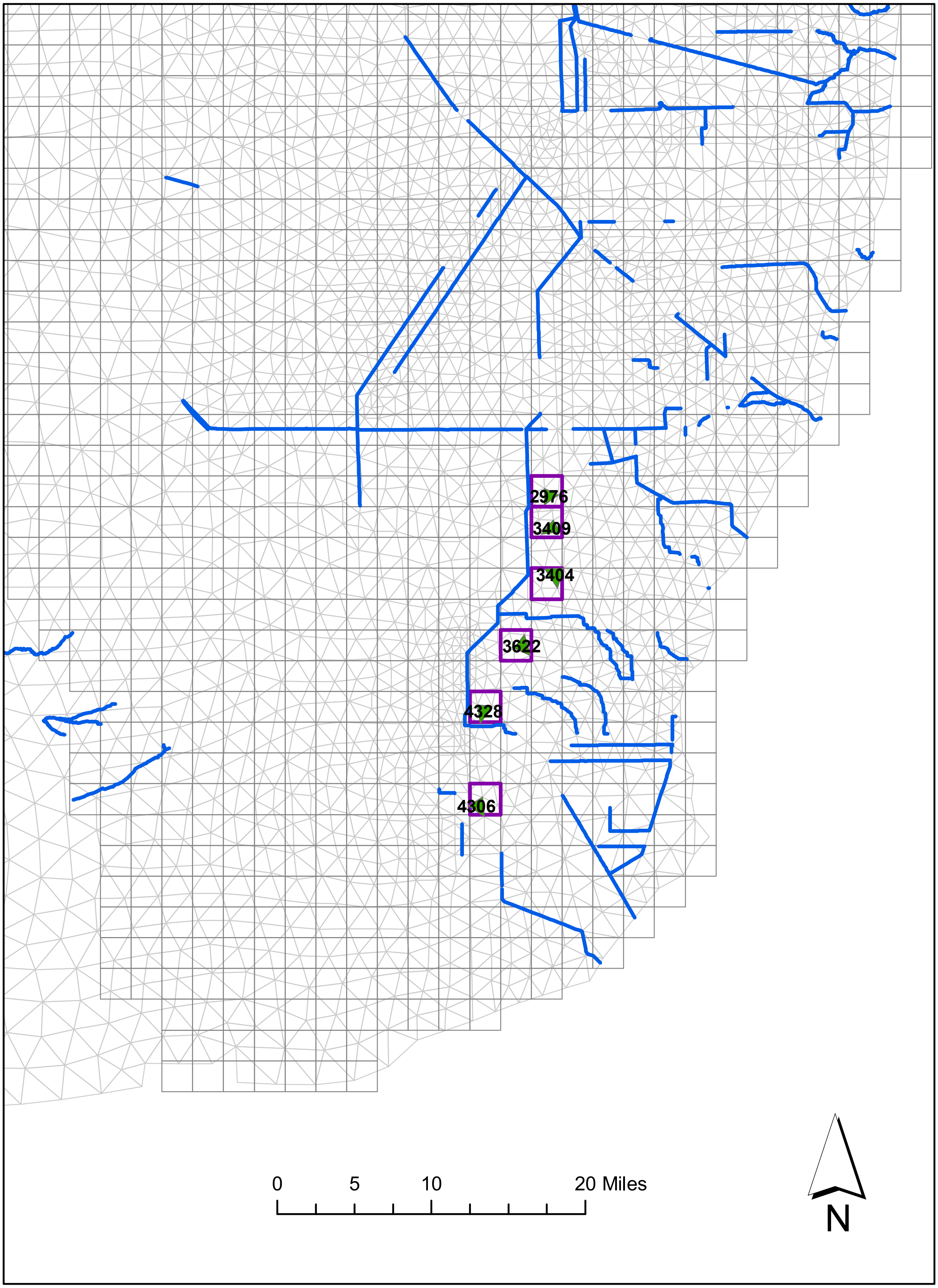
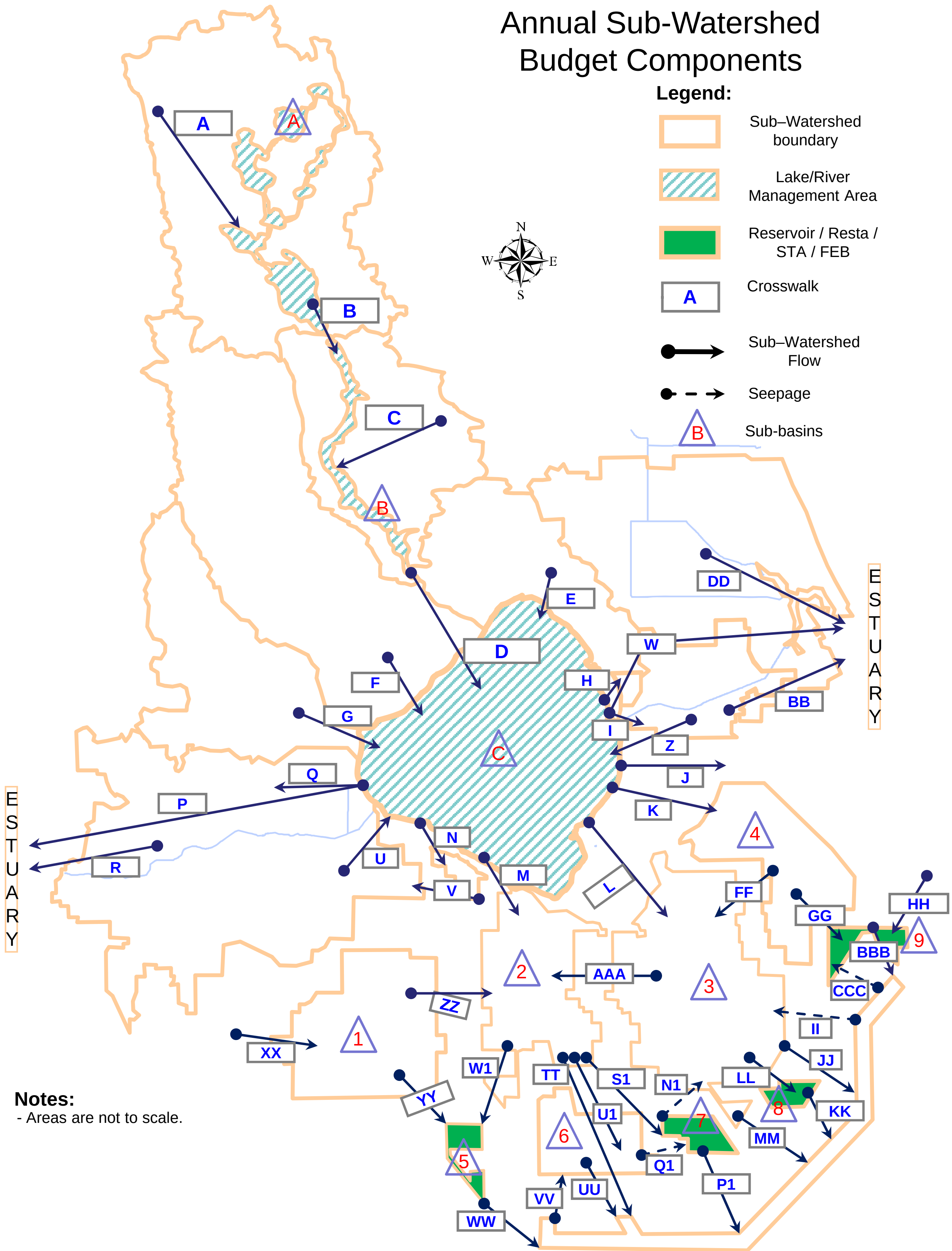


ANNEX A-2: REFERENCE 4

RSM-BN WATER BUDGET MAPS FOR BASELINES AND ALT4R2

RSMBN ECB & 2012EC

Annual Sub-Watershed Budget Components



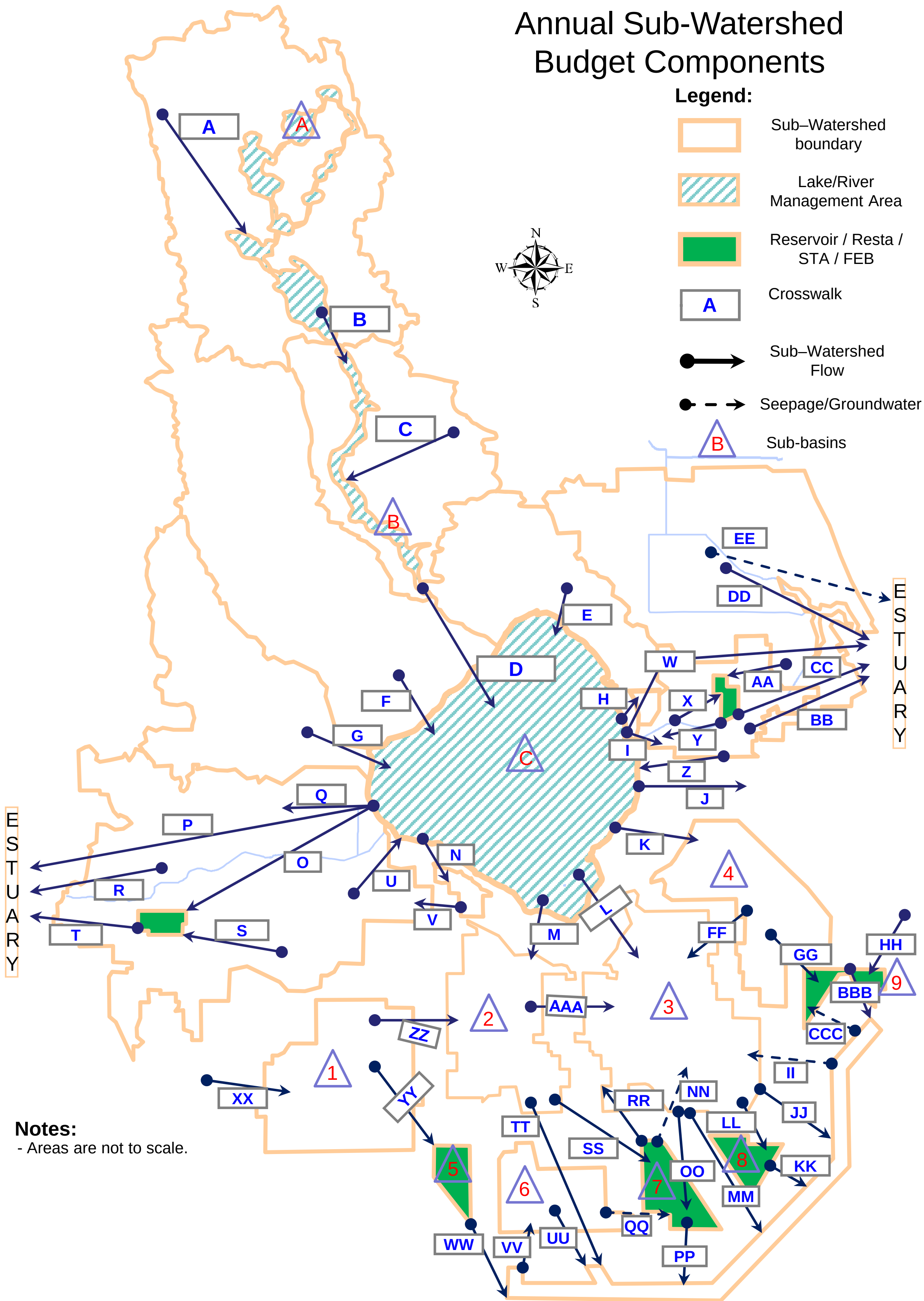
Notes:
- Areas are not to scale.

RSMBN ECB & 2012EC

A	**Sub-basins (Red Text on Map)**
A	Upper Kissimmee Lake Management Areas
B	Lower Kissimmee River Management Areas
C	Lake Okeechobee
1	C-139
2	EAA Miami
3	EAA NNR-Hillsboro
4	EAA WPB
5	STA5&6
6	Rotenberger& Holeyland
7	STA3/4
8	STA2
9	STA1W&1E

RSMBN IORBL1

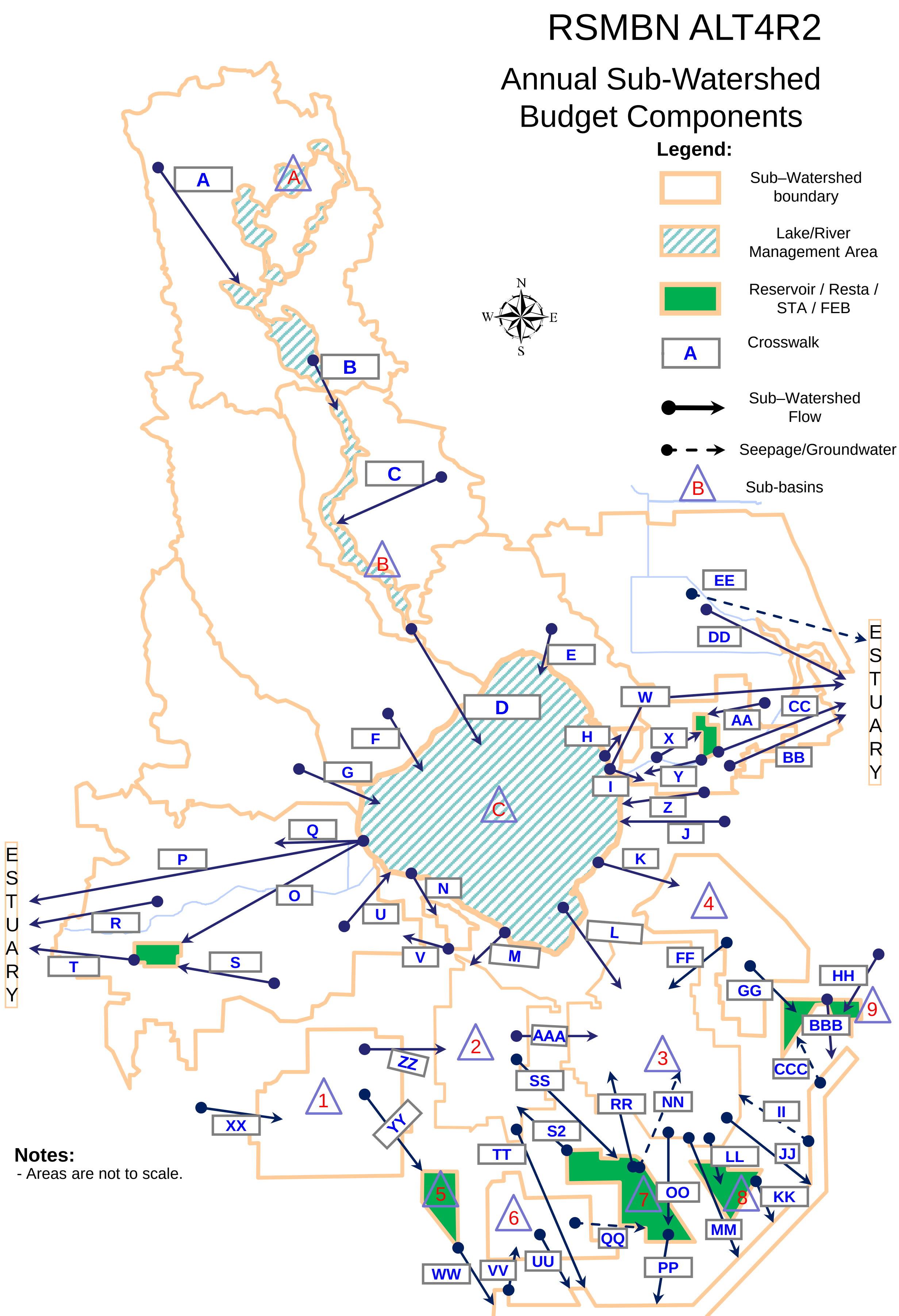
Annual Sub-Watershed Budget Components



Notes:
- Areas are not to scale.

RSMBN IORBL1

A	**Sub-basins (Red Text on Map)**
A	Upper Kissimmee Lake Management Areas
B	Lower Kissimmee River Management Areas
C	Lake Okeechobee
1	C-139
2	EAA Miami
3	EAA NNR-Hillsboro
4	EAA WPB
5	STA5&6
6	Rotenberger& Holeyland
7	STA3/4 & A1FEB
8	STA2
9	STA1W&1E



Notes:
- Areas are not to scale.

RSMBN ALT4R2

	**Sub-basins (Red Text on Map)**
A	Upper Kissimmee Lake Management Areas
B	Lower Kissimmee River Management Areas
C	Lake Okeechobee
1	C-139
2	EAA Miami
3	EAA NNR-Hillsboro
4	EAA WPB
5	STA5&6
6	Rotenberger& Holeyland
7	STA3/4 & FEB
8	STA2
9	STA1W&1E

RSMBN ECB/2012EC/IORBL1/ALT4R2 Crosswalk.

The following letters correspond to blue text in rectangles on maps. A

A. Upper Kissimmee basin total watershed flows.

B. Lake Kissimmee total flows.

C. Lower Kissimmee basin total watershed flows.

D. S65E total flows to Lake Okeechobee.

E. Net total flows from Taylor Creek/Nubbin Slough basin, Northeast Lake Shore & North Lake Shore to Lake Okeechobee.

F. Net total flows from Istokpoga/Brighton to Lake Okeechobee.

G. Fisheating creek runoff to Lake Okeechobee.

H. Lake Okeechobee flows to FPL reservoir.

I. Total Lake Okeechobee flows to C44 basin.

J. Total net Lake Okeechobee flows to L8 canal/basin.

K. Total net Lake Okeechobee flows to West Palm Beach basin.

L. Total net Lake Okeechobee flows to North New River/Hillsboro basin.

M. Total net Lake Okeechobee flows to Miami basin.

N. Total Lake Okeechobee flows to S4 basin.

O. Lake Okeechobee flows to C43 Reservoir.

P. Total net Lake Okeechobee flows to Caloosahatchee estuary.

Q. Lake Okeechobee flows to C43 Basin.

R. C43 basin flows to Caloosahatchee estuary.

S. C43 basin flows to C43 Reservoir.

T. C43 Reservoir flows to Caloosahatchee Estuary.

U. C43 basin flows to Lake Okeechobee.

V. S4 basin flows to C43 basin.

W. Net Lake Okeechobee flows to St. Lucie estuary.

X. C44 basin flows to C44 reservoir/sta.

Y. C44 reservoir/sta flows to C44 basin.

Z. C44 Basin flows to Lake Okeechobee.

AA. C23 basin flows (non-C44 basin) to C44 reservoir/sta.

BB. C44 basin flows to St. Lucie estuary.

CC. C44 reservoir/sta to St. Lucie estuary.

DD. Non-C44 basins (Runoff from tributaries of the St. Lucie Estuaries) to St. Lucie estuary.

EE. Non-C44 basins Groundwater to St. Lucie estuary.

FF. West Palm Beach basin flows to North New River basin.

GG. West Palm Beach basin flows to STA1W.

HH. Net L8/C51/LWDD flows to STA1E.

II. L7/L6 canal flows to North New River/Hillsboro basin.

JJ. North New River/Hillsboro basin flows to L7 canal.

KK. STA2 flows to L6 canal.

LL. North New River/Hillsboro basin flows to STA2.

MM. North New River/Hillsboro basin flows to L6 canal.

NN. STA34&A1FEB(or FEB) flows to North New River/Hillsboro basin.

N1. STA34 flows to North New River/Hillsboro basin (ECB).

OO. North New River/Hillsboro flows basin to STA34&A1FEB(or FEB).

PP. STA34&A1FEB(or FEB) flows to Miami/L4/L5 canal.

P1. STA34 flows to Miami/L4/L5 canal(ECB).

QQ. Rotenberger & Holeyland flows to STA34&A1FEB(or FEB).

Q1. Rotenberger & Holeyland flows to STA34(ECB)

RR. STA34&A1FEB (or FEB) flows to North New River/Hillsboro basin.

SS. Miami basin flows to STA34&A1FEB(or FEB).

S1. Miami basin flows to STA34 (ECB).

S2. STA34&FEB flows to Miami basin(ALT4R1).

TT. Miami basin flows to Lower Miami/L4/L5 canal.

UU. Rotenberger & Holeyland flows to Miami/L4/L5 canal.

U1. Miami basin flows to Rotenberger & Holeyland. (ECB)

VV. Miami/L4/L5 canal flows to Rotenberger & Holeyland.

WW. STA56 flows to Miami/L4/L5 canal.

W1. Miami basin flows to STA56.

XX. Offsite water supply/recharge flows to C139 basin.

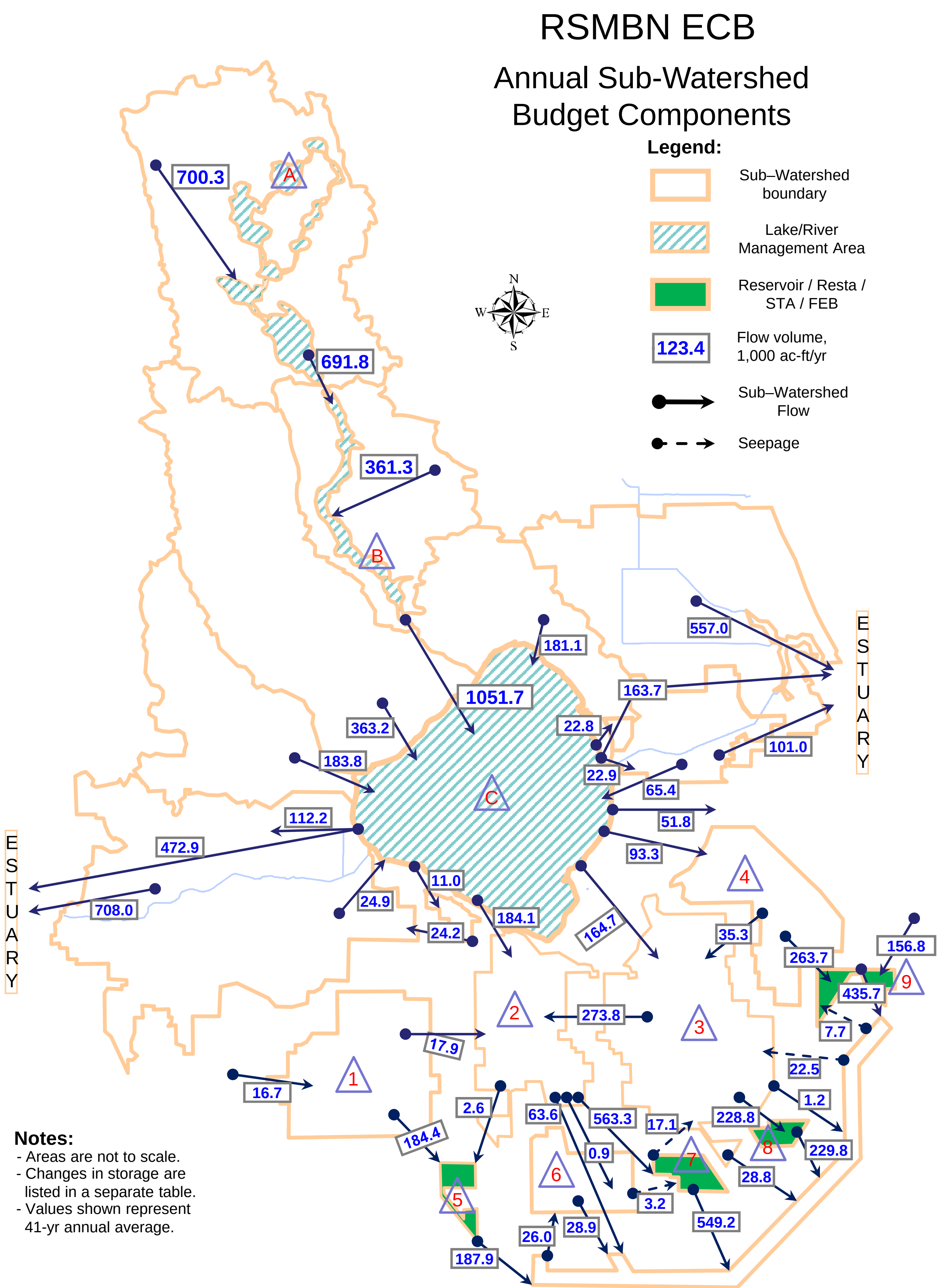
YY. C139 basin flows to STA56.

ZZ. C139 basin flows to Miami basin.

AAA. Miami basin flows to North New River/Hillsboro basin.

BBB. STA1W&STA1E flows to L7 canal.

CCC. L7 canal flows to STA1W.



RSMBN ECB
Annual Sub-Watershed Budget Components
Legend:
Sub–Watershed boundary
Lake/River Management Area
Reservoir / Resta / STA / FEB
123.4
Flow volume, 1,000 ac-ft/yr
Sub–Watershed Flow
Seepage
N
W
E
S
ESTUARY
ESTUARY
A
B
C
1
2
3
4
5
6
7
8
9
700.3
691.8
361.3
1051.7
181.1
557.0
163.7
363.2
183.8
22.8
22.9
101.0
65.4
51.8
112.2
472.9
708.0
93.3
11.0
24.9
24.2
184.1
164.7
35.3
263.7
156.8
435.7
7.7
273.8
17.9
22.5
16.7
2.6
63.6
563.3
17.1
228.8
1.2
184.4
0.9
229.8
28.8
3.2
549.2
26.0
28.9
187.9
Notes:
- Areas are not to scale.
- Changes in storage are listed in a separate table.
- Values shown represent 41-yr annual average.

RSMBN ECB

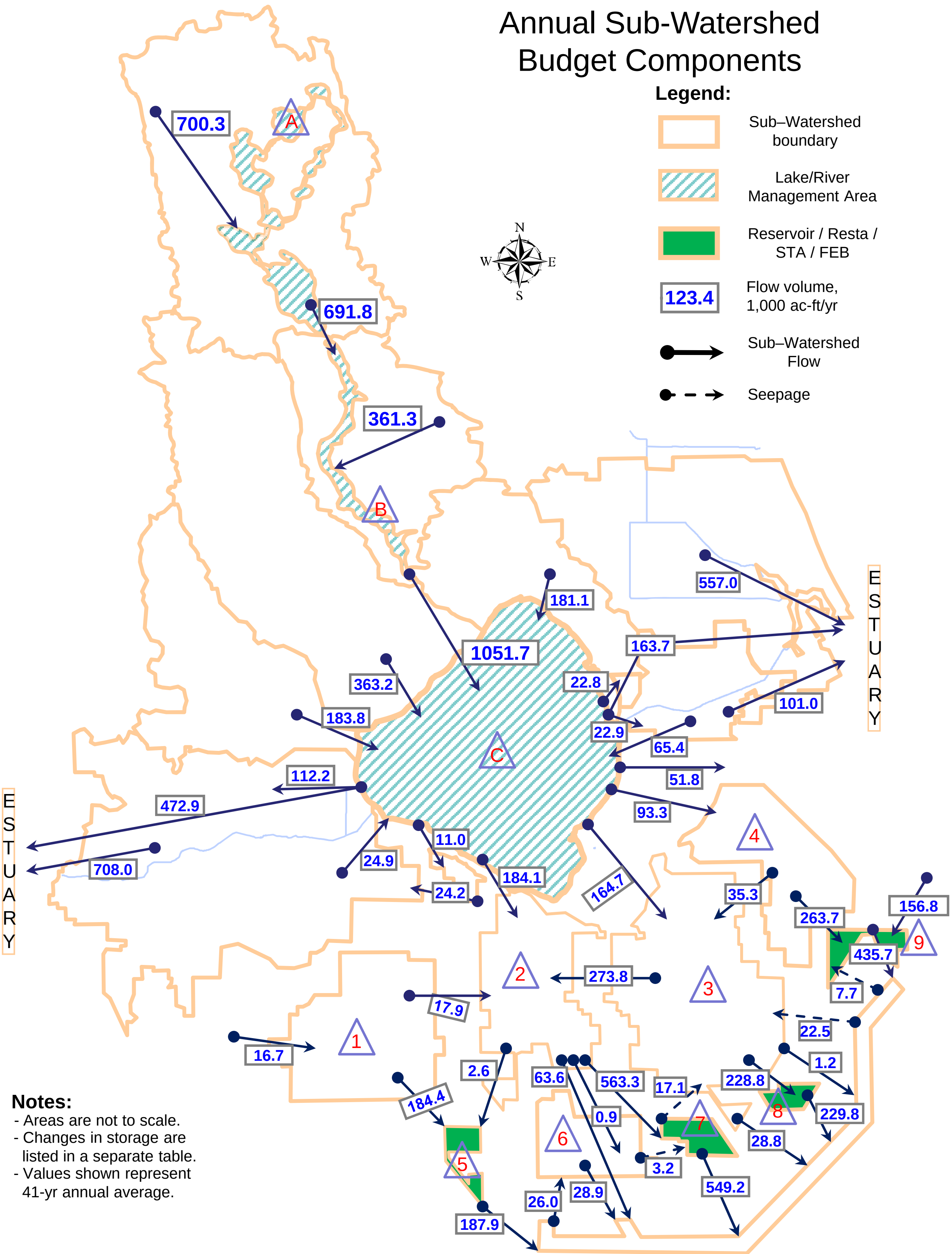
		RF	ET	Change in Storage	Residual
A	Upper Kissimmee Lake Management Areas	370.0	380.9	-2.4	0.0
B	Lower Kissimmee River Management Areas	36.5	37.7	0.2	0.0
C	Lake Okeechobee*	1643.0	2096.7	17.2	0.0
1	C-139	676.6	479.6	11.4	0.0
2	EAA Miami	554.8	400.3	-0.1	0.0
3	EAA NNR-Hillsboro	1037.6	744.5	0.1	0.0
4	EAA WPB	539.7	334.0	-0.1	0.1
5	STA5&6	41.7	40.9	0.0	0.0
6	Rotenberger & Holeyland	275.0	268.5	1.2	0.1
7	STA3/4	73.5	73.6	0.1	0.0
8	STA2	41.5	40.5	0.0	0.0
9	STA1W&1E	62.0	54.6	-0.1	0.1

Table: Rainfall, evapotranspiration and change in storage volumes (1,000 ac-ft/yr) for major sub-watersheds simulated in RSMBN

note*: Lake Okeechobee MDS term = -99.8 kaf/yr

RSMBN 2012EC

Annual Sub-Watershed Budget Components



Notes:
- Areas are not to scale.
- Changes in storage are listed in a separate table.
- Values shown represent 41-yr annual average.

RSMBN 2012EC

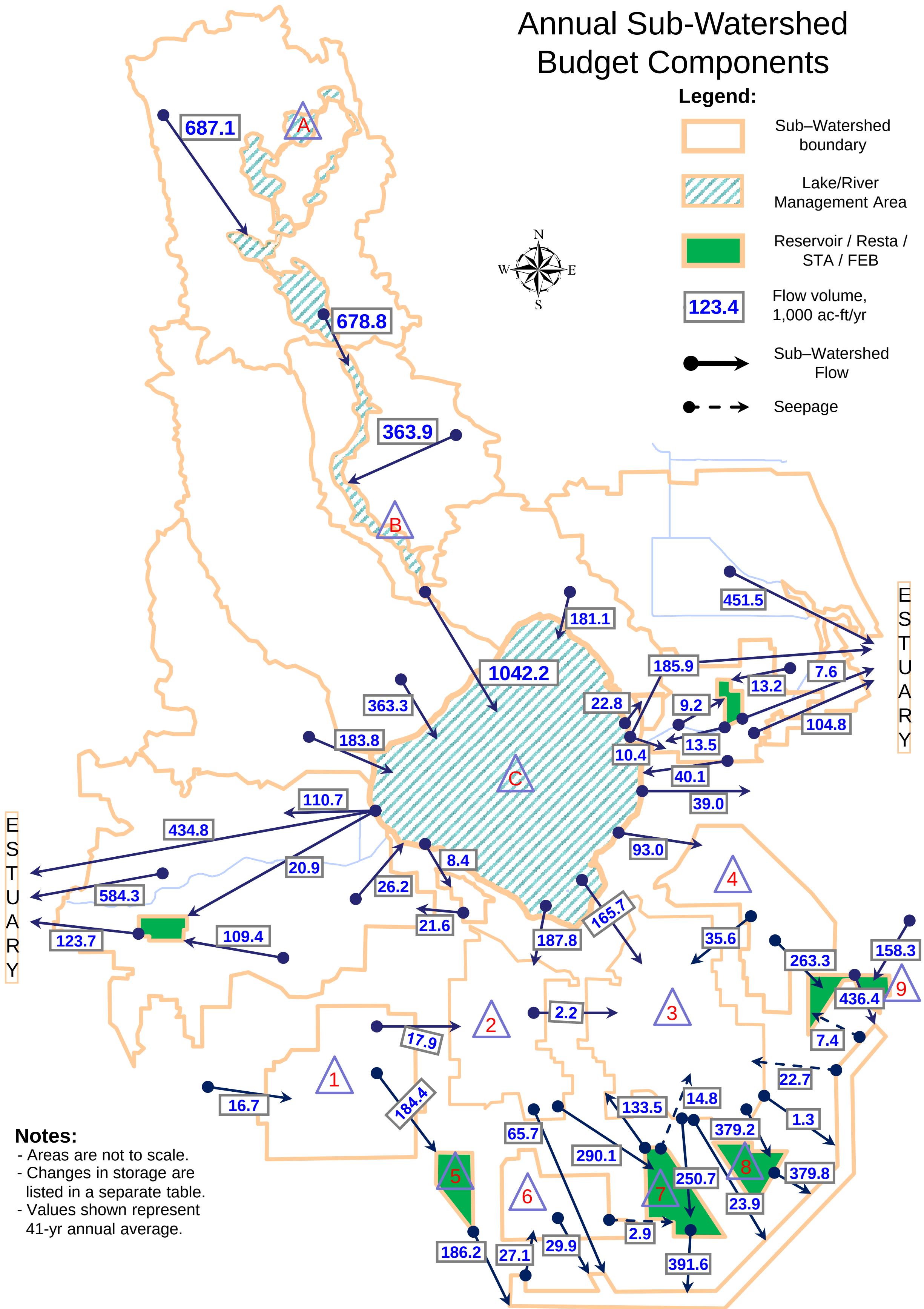
		RF	ET	Change in Storage	Residual
A	Upper Kissimmee Lake Management Areas	370.0	380.9	-2.4	0.0
B	Lower Kissimmee River Management Areas	36.5	37.7	0.2	0.0
C	Lake Okeechobee*	1643.0	2096.7	17.2	0.0
1	C-139	676.6	479.6	11.4	0.0
2	EAA Miami	554.8	400.3	-0.1	0.0
3	EAA NNR-Hillsboro	1037.6	744.5	0.1	0.0
4	EAA WPB	539.7	334.0	-0.1	0.1
5	STA5&6	41.7	40.9	0.0	0.0
6	Rotenberger & Holeyland	275.0	268.5	1.2	0.1
7	STA3/4	73.5	73.6	0.1	0.0
8	STA2	41.5	40.5	0.0	0.0
9	STA1W&1E	62.0	54.6	-0.1	0.1

Table: Rainfall, evapotranspiration and change in storage volumes (1,000 ac-ft/yr) for major sub-watersheds simulated in RSMBN

note*: Lake Okeechobee MDS term = -99.8 kaf/yr

RSMBN FWO

Annual Sub-Watershed Budget Components



Notes:
- Areas are not to scale.
- Changes in storage are listed in a separate table.
- Values shown represent 41-yr annual average.

RSMBN FWO

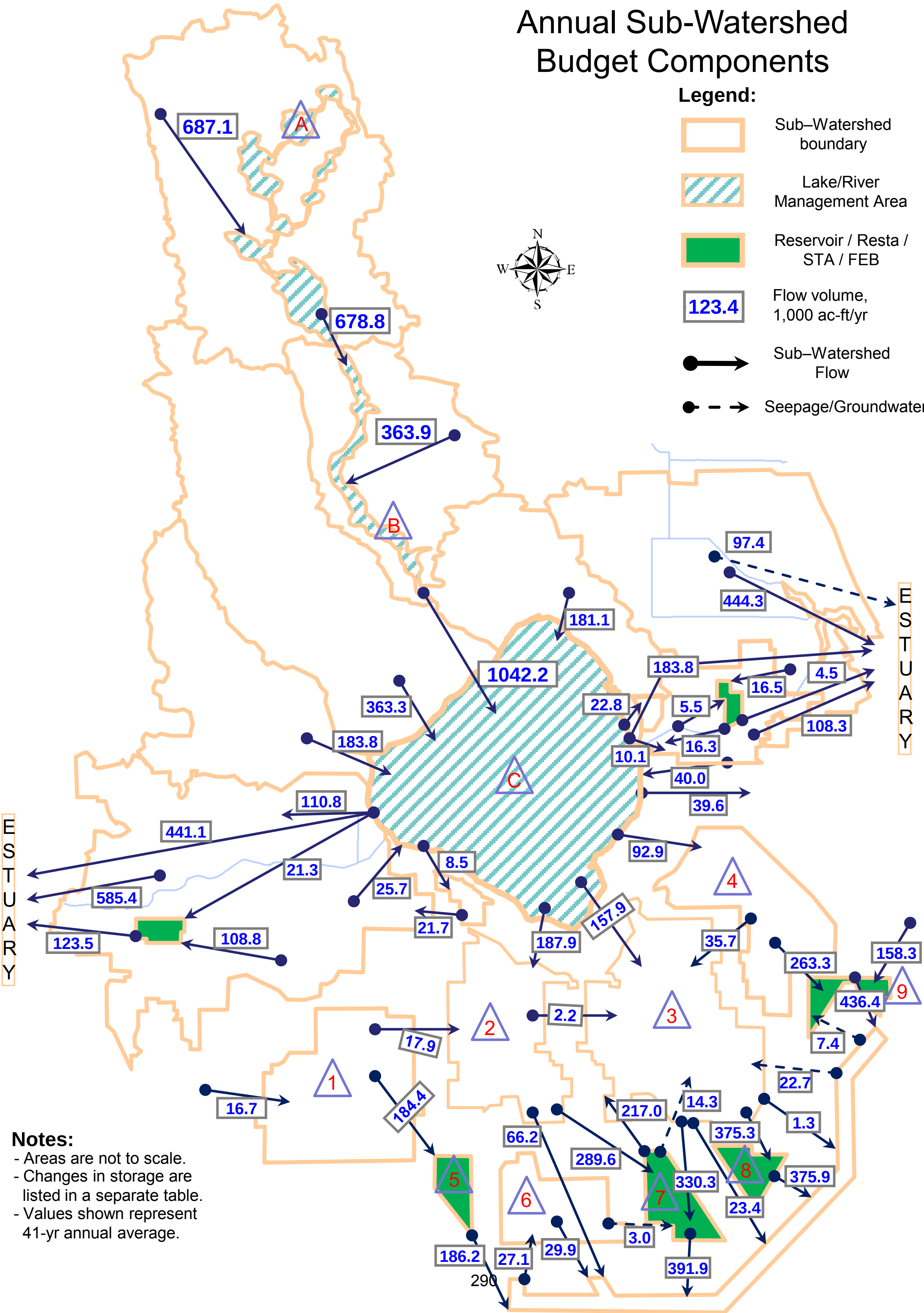
		RF	ET	Change in Storage	Residual
A	Upper Kissimmee Lake Management Areas	405.1	414.0	-0.6	0.0
B	Lower Kissimmee River Management Areas	22.9	23.4	0.0	0.0
C	Lake Okeechobee*	1643.0	2085.1	15.3	0.1
1	C-139	676.6	479.6	11.4	0.0
2	EAA Miami	527.7	375.5	0.0	-0.1
3	EAA NNR-Hillsboro	937.1	656.6	-0.1	0.0
4	EAA WPB	539.7	333.8	-0.1	0.1
5	STA5&6	68.7	66.9	0.0	0.0
6	Rotenberger & Holeyland	275.0	268.2	1.1	0.0
7	STA3/4 & A1FEB	139.8	142.6	1.1	-0.1
8	STA2	75.0	74.4	0.1	-0.1
9	STA1W&1E	62.0	54.6	-0.1	0.1

Table: Rainfall, evapotranspiration and change in storage volumes (1,000 ac-ft/yr) for major sub-watersheds simulated in RSMBN

note*: Lake Okeechobee MDS term = -99.8 kaf/yr

RSMBN IORBL1

Annual Sub-Watershed Budget Components



Notes:
- Areas are not to scale.
- Changes in storage are listed in a separate table.
- Values shown represent 41-yr annual average.

RSMBN IORBL1

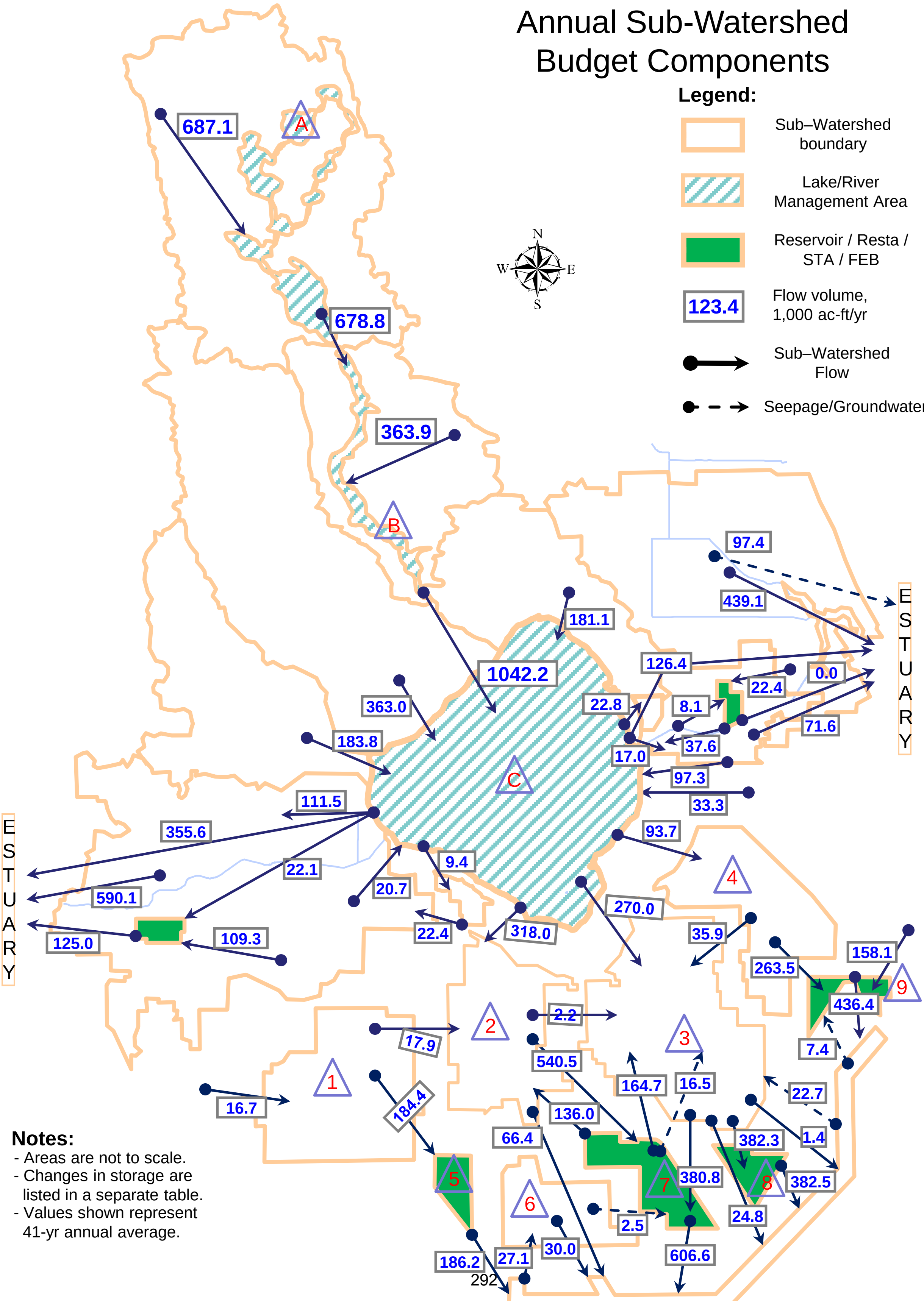
		RF	ET	Change in Storage	Residual
A	Upper Kissimmee Lake Management Areas	405.1	414.0	-0.6	0.0
B	Lower Kissimmee River Management Areas	22.9	23.4	0.0	0.0
C	Lake Okeechobee*	1643.0	2086.0	15.7	0.2
1	C-139	676.6	479.6	11.4	0.0
2	EAA Miami	527.7	375.5	0.0	0.1
3	EAA NNR-Hillsboro	937.1	656.6	-0.1	-0.2
4	EAA WPB	539.7	333.8	0.0	-0.1
5	STA5&6	68.7	66.9	0.0	0.0
6	Rotenberger & Holeyland	275.0	268.1	1.1	0.0
7	STA3/4 & A1FEB	139.8	139.1	0.3	0.0
8	STA2	75.0	74.4	0.1	0.0
9	STA1W&1E	62.0	54.6	-0.1	0.0

Table: Rainfall, evapotranspiration and change in storage volumes (1,000 ac-ft/yr) for major sub-watersheds simulated in RSMBN

note*: Lake Okeechobee MDS term = -99.8 kaf/yr

RSMBN ALT4R2

Annual Sub-Watershed Budget Components



Notes:
- Areas are not to scale.
- Changes in storage are listed in a separate table.
- Values shown represent 41-yr annual average.

RSMBN ALT4R2

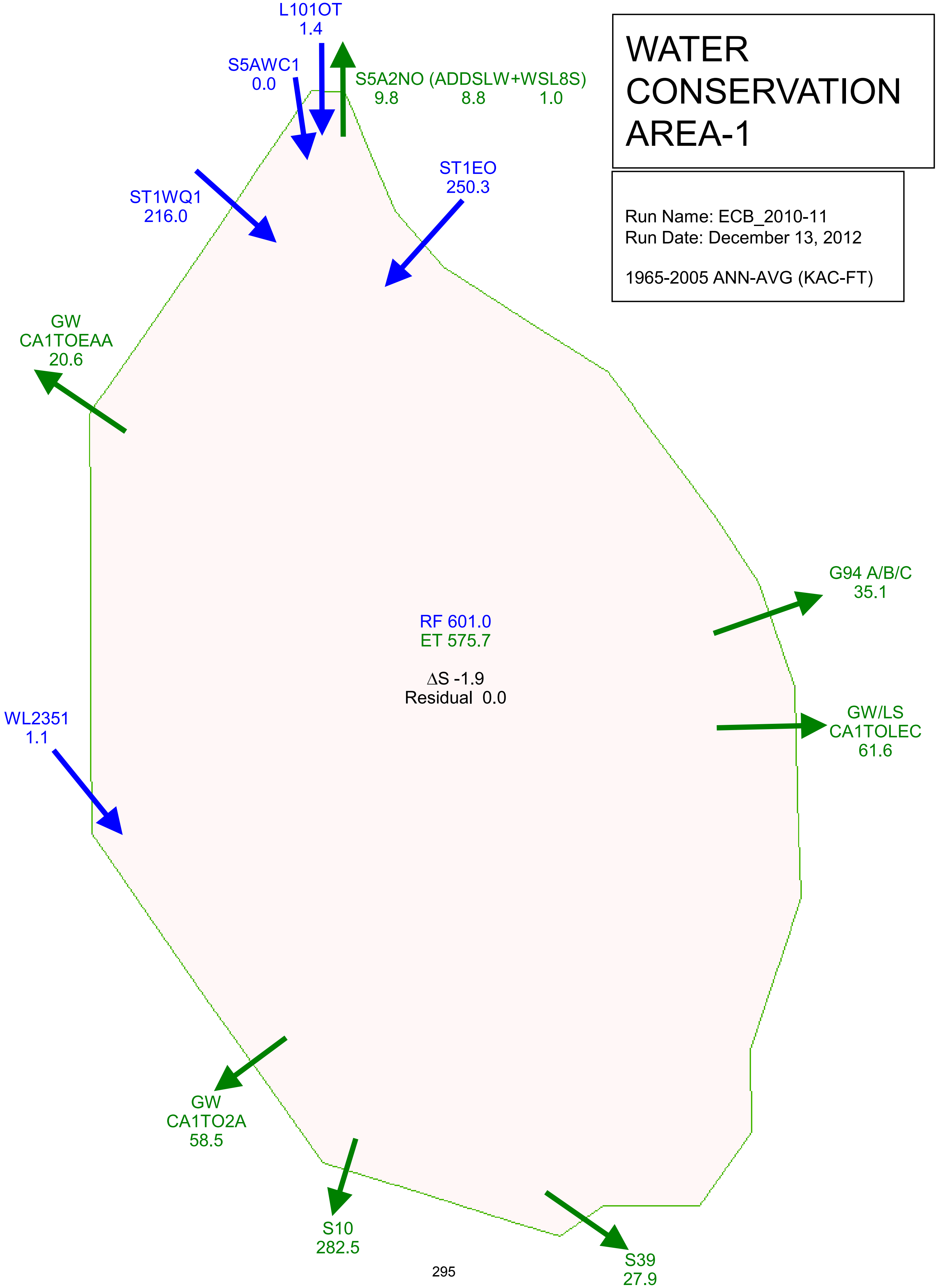
		RF	ET	Change in Storage	Residual
A	Upper Kissimmee Lake Management Areas	405.1	414.0	-0.6	0.0
B	Lower Kissimmee River Management Areas	22.9	23.4	-0.1	0.0
C	Lake Okeechobee*	1643.0	2101.6	21.5	0.0
1	C-139	676.6	479.6	11.4	0.0
2	EAA Miami	494.3	352.8	0.0	0.0
3	EAA NNR-Hillsboro	905.4	640.7	-0.1	0.6
4	EAA WPB	539.7	334.0	0.1	-0.1
5	STA5&6	68.7	66.9	0.0	0.0
6	Rotenberger & Holeyland	275.0	268.4	1.1	0.0
7	STA3/4 & FEB	197.4	195.6	1.2	0.0
8	STA2	75.0	74.6	0.2	0.0
9	STA1W&1E	62.0	54.6	-0.1	0.0

Table: Rainfall, evapotranspiration and change in storage volumes (1,000 ac-ft/yr) for major sub-watersheds simulated in RSMBN

note*: Lake Okeechobee MDS term = -99.8 kaf/yr

ANNEX A-2: REFERENCE 5

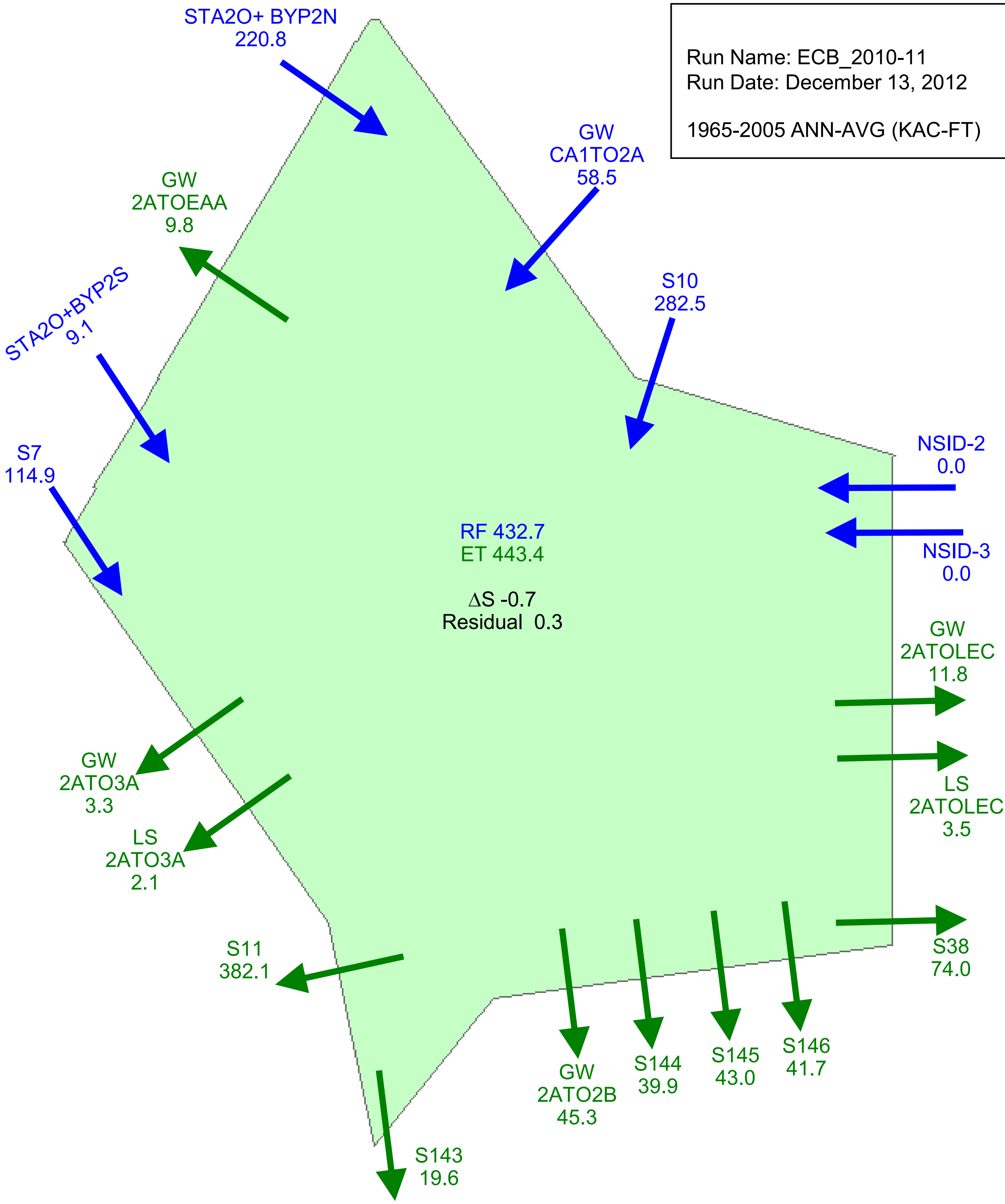
RSM-GL WATER BUDGET MAPS FOR BASELINES AND ALT4R2


WATER CONSERVATION AREA-1
Run Name: ECB_2010-11
Run Date: December 13, 2012
1965-2005 ANN-AVG (KAC-FT)
L101OT
1.4
S5AWC1
0.0
S5A2NO (ADDSLW+WSL8S)
9.8 8.8 1.0
ST1WQ1
216.0
ST1EO
250.3
GW
CA1TOEAA
20.6
G94 A/B/C
35.1
RF 601.0
ET 575.7
ΔS -1.9
Residual 0.0
WL2351
1.1
GW/LS
CA1TOLEC
61.6
GW
CA1TO2A
58.5
S10
282.5
S39
27.9

WATER CONSERVATION AREA-2A

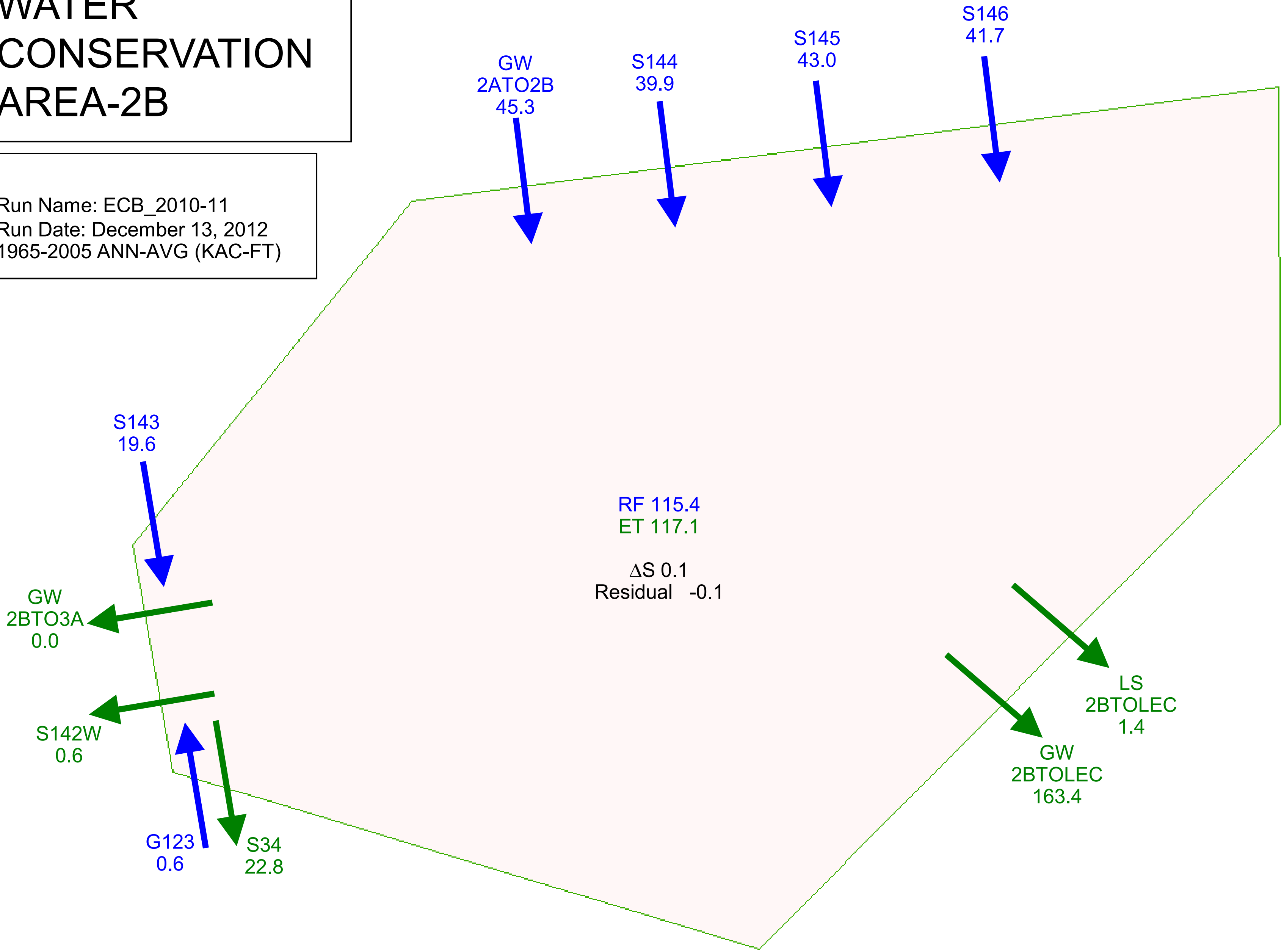
Run Name: ECB_2010-11
Run Date: December 13, 2012

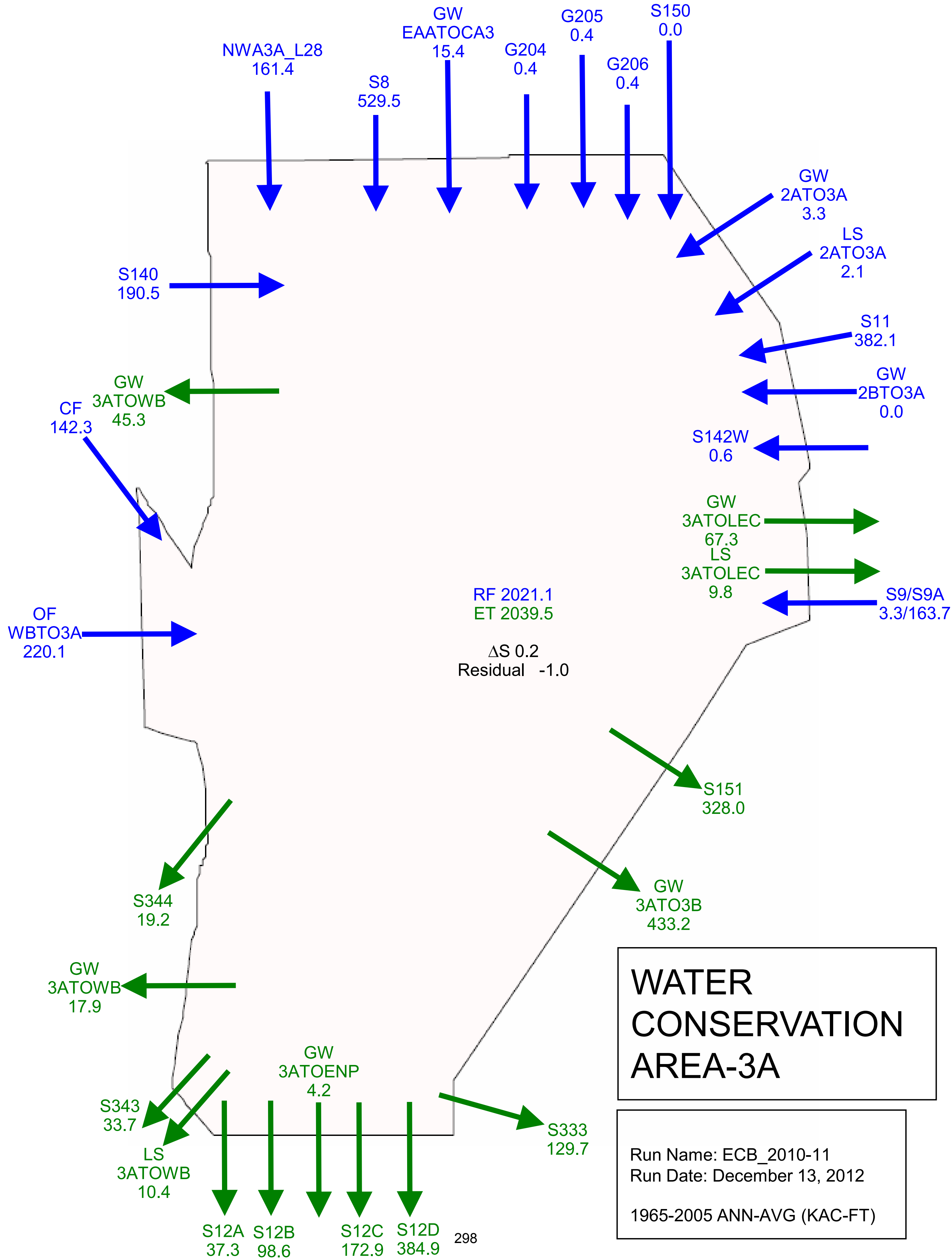
1965-2005 ANN-AVG (KAC-FT)



WATER CONSERVATION AREA-2B

Run Name: ECB_2010-11
Run Date: December 13, 2012
1965-2005 ANN-AVG (KAC-FT)

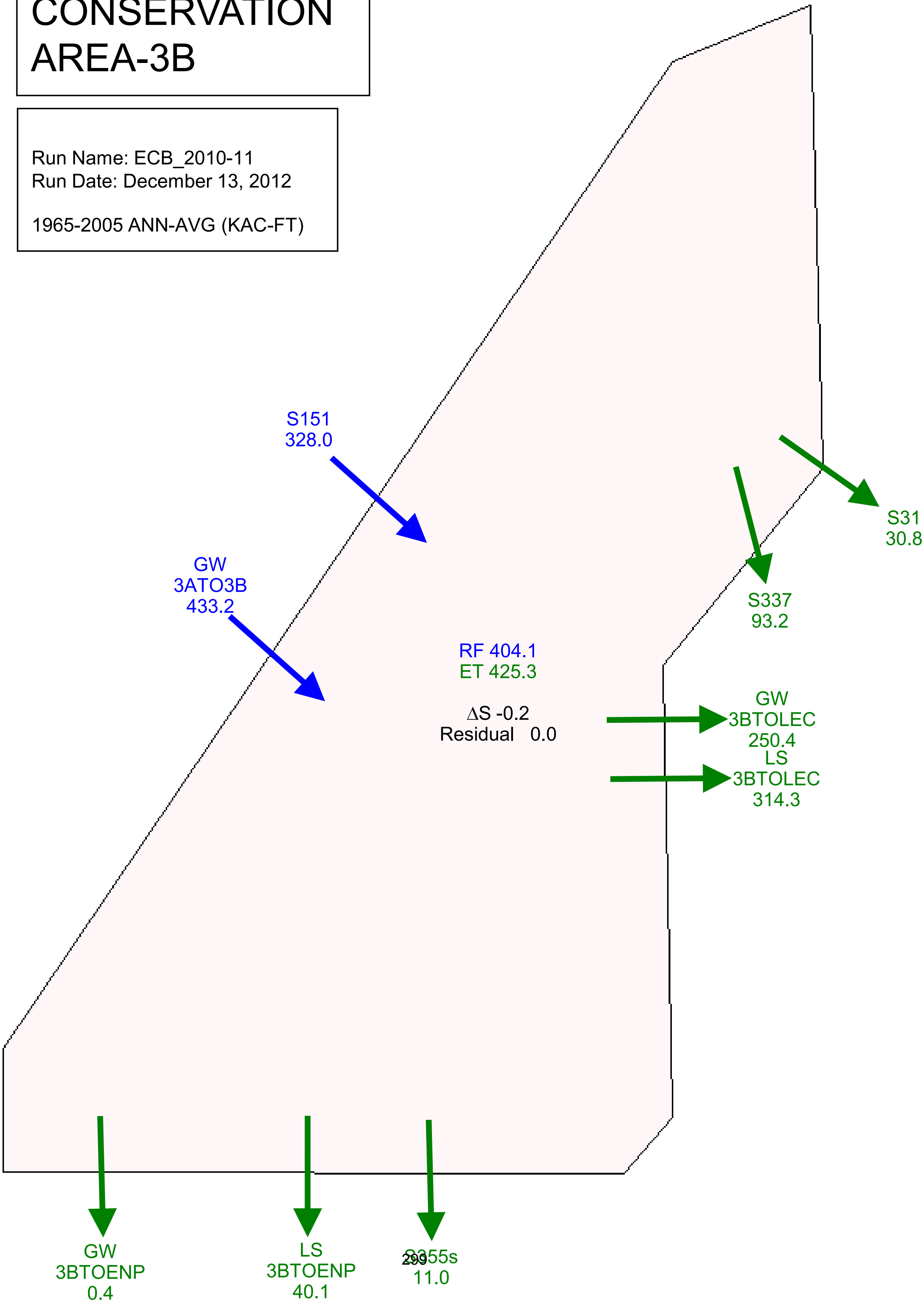



NWA3A_L28
161.4
S8
529.5
GW
EAATOCA3
15.4
G204
0.4
G205
0.4
G206
0.4
S150
0.0
GW
2ATO3A
3.3
LS
2ATO3A
2.1
S11
382.1
GW
2BTO3A
0.0
S142W
0.6
GW
3ATOLEC
67.3
LS
3ATOLEC
9.8
S9/S9A
3.3/163.7
S140
190.5
GW
3ATOWB
45.3
CF
142.3
OF
WBTO3A
220.1
RF 2021.1
ET 2039.5
ΔS 0.2
Residual -1.0
S151
328.0
GW
3ATO3B
433.2
S344
19.2
GW
3ATOWB
17.9
GW
3ATOENP
4.2
S343
33.7
LS
3ATOWB
10.4
S12A
37.3
S12B
98.6
S12C
172.9
S12D
384.9
S333
129.7
WATER
CONSERVATION
AREA-3A
Run Name: ECB_2010-11
Run Date: December 13, 2012
1965-2005 ANN-AVG (KAC-FT)

WATER CONSERVATION AREA-3B

Run Name: ECB_2010-11
Run Date: December 13, 2012

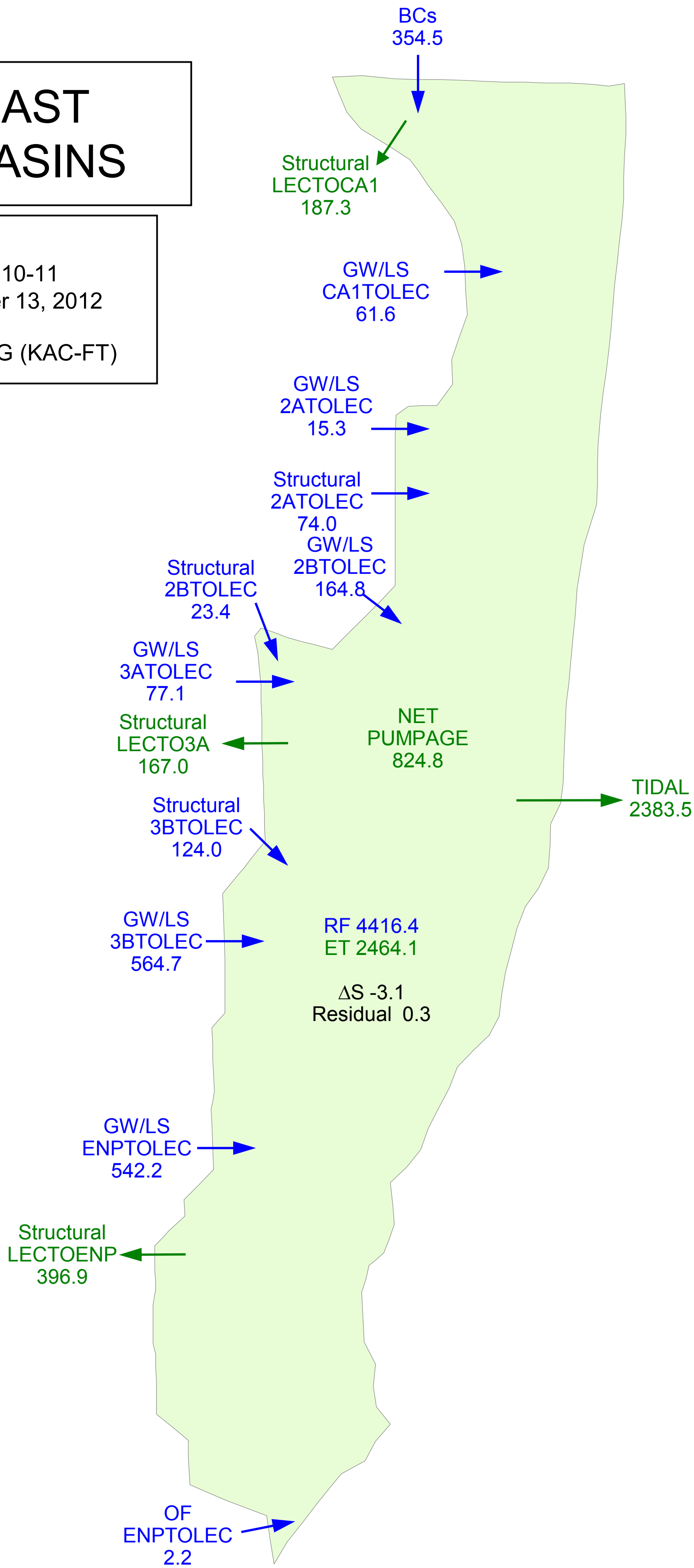
1965-2005 ANN-AVG (KAC-FT)

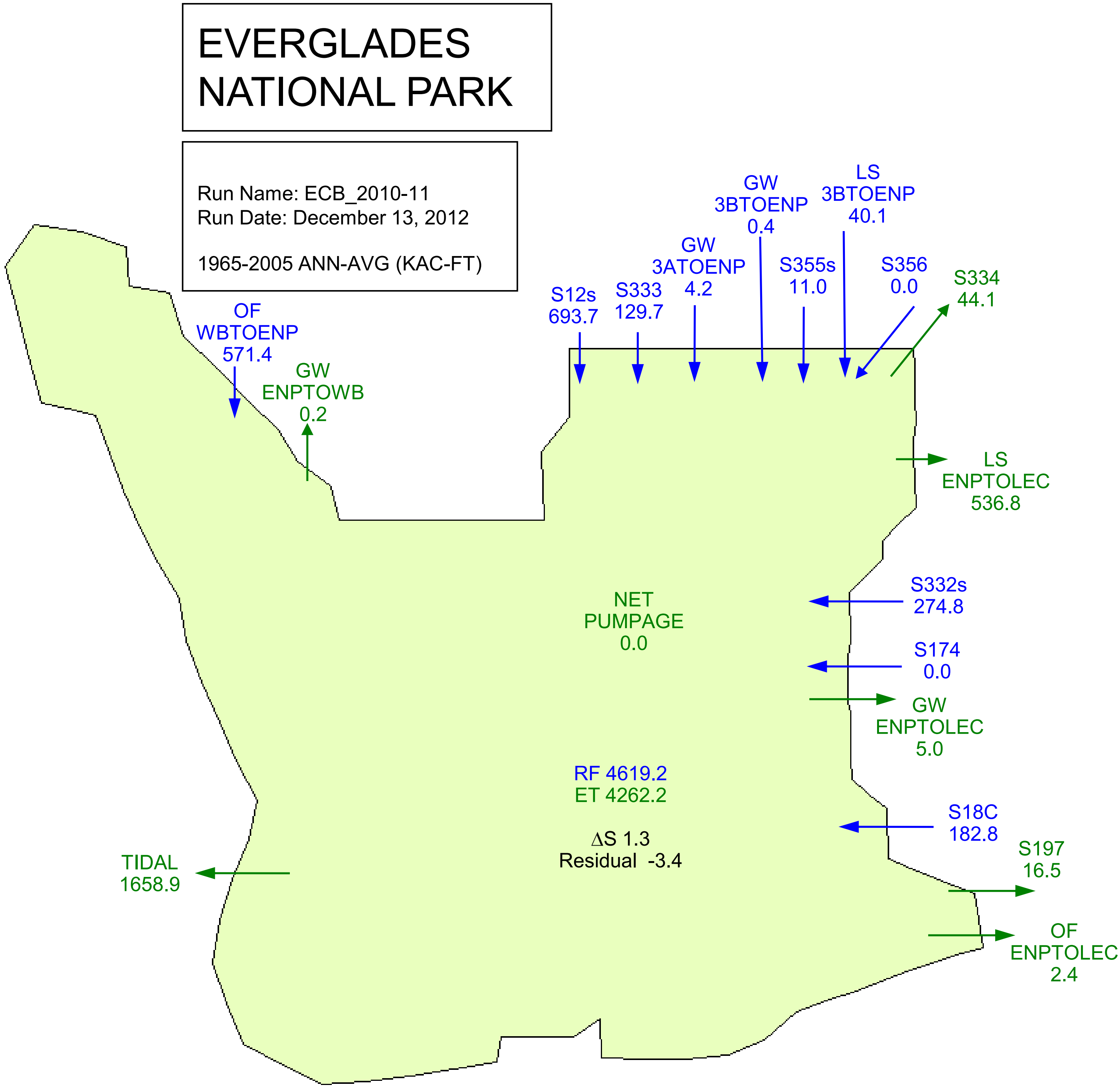


LOWER EAST COAST BASINS

Run Name: ECB_2010-11
Run Date: December 13, 2012

1965-2005 ANN-AVG (KAC-FT)

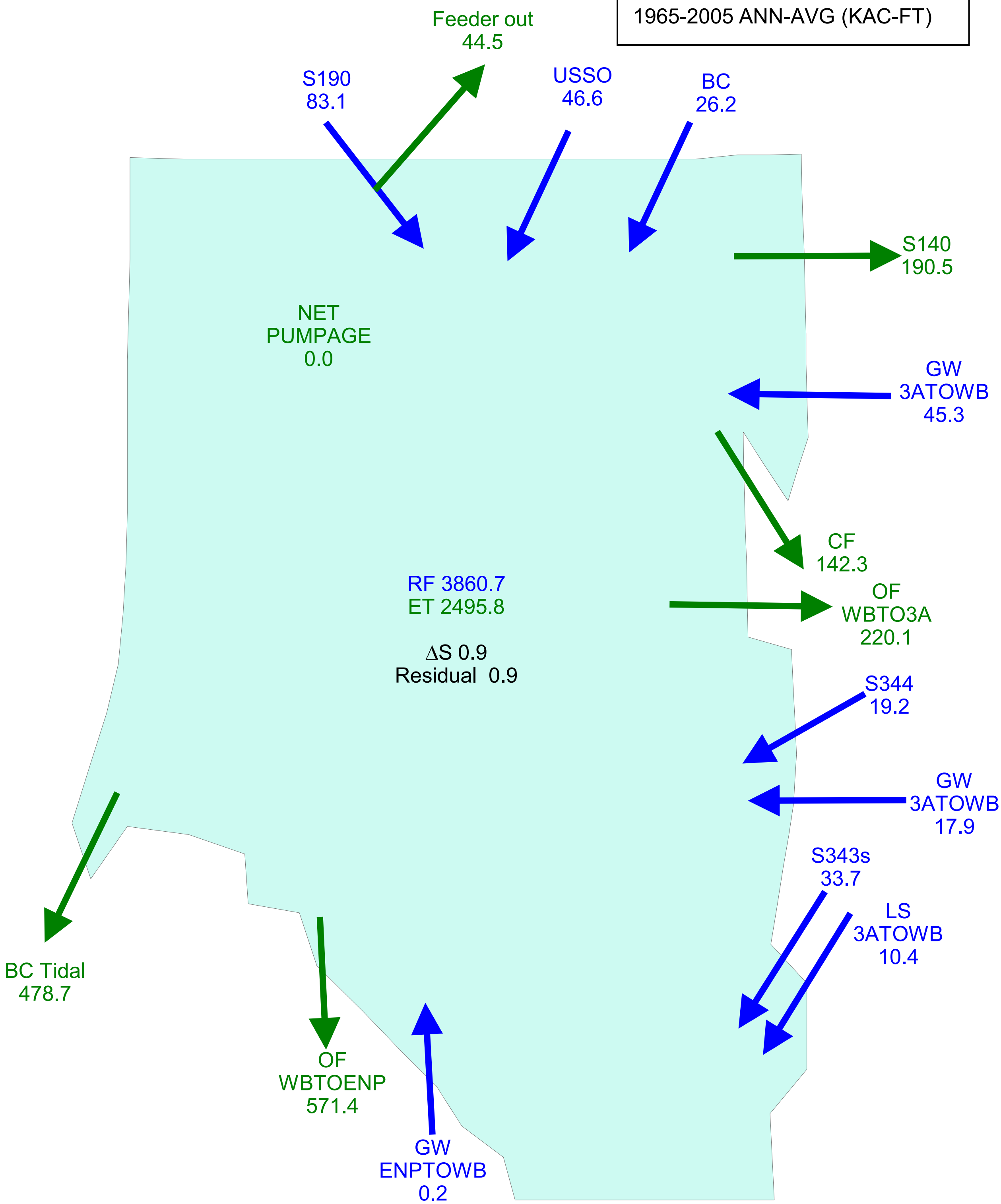



EVERGLADES
NATIONAL PARK
Run Name: ECB_2010-11
Run Date: December 13, 2012
1965-2005 ANN-AVG (KAC-FT)
OF
WBTOENP
571.4
GW
ENPTOWB
0.2
S12s
693.7
S333
129.7
GW
3ATOENP
4.2
GW
3BTOENP
0.4
S355s
11.0
LS
3BTOENP
40.1
S356
0.0
S334
44.1
LS
ENPTOLEC
536.8
S332s
274.8
NET
PUMPAGE
0.0
S174
0.0
GW
ENPTOLEC
5.0
RF 4619.2
ET 4262.2
ΔS 1.3
Residual -3.4
S18C
182.8
S197
16.5
TIDAL
1658.9
OF
ENPTOLEC
2.4

WESTERN BASINS

Run Name: ECB_2010-11
Run Date: December 13, 2012

1965-2005 ANN-AVG (KAC-FT)

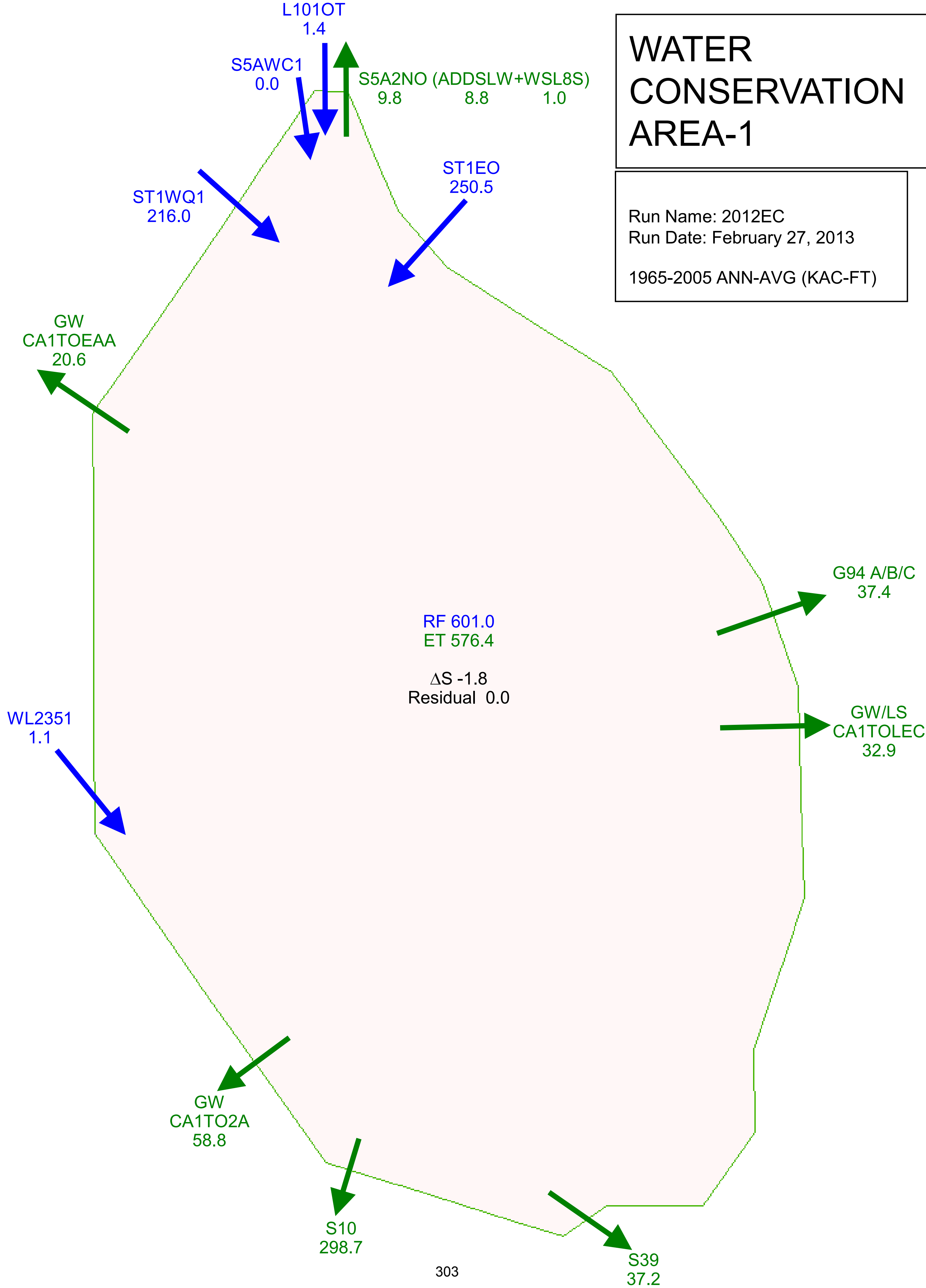


Note: Western Basins (WB) constitute L-28 Interceptor, Feeder Canal, L-28 Gap and East Collier Basins

WATER CONSERVATION AREA-1

Run Name: 2012EC
Run Date: February 27, 2013

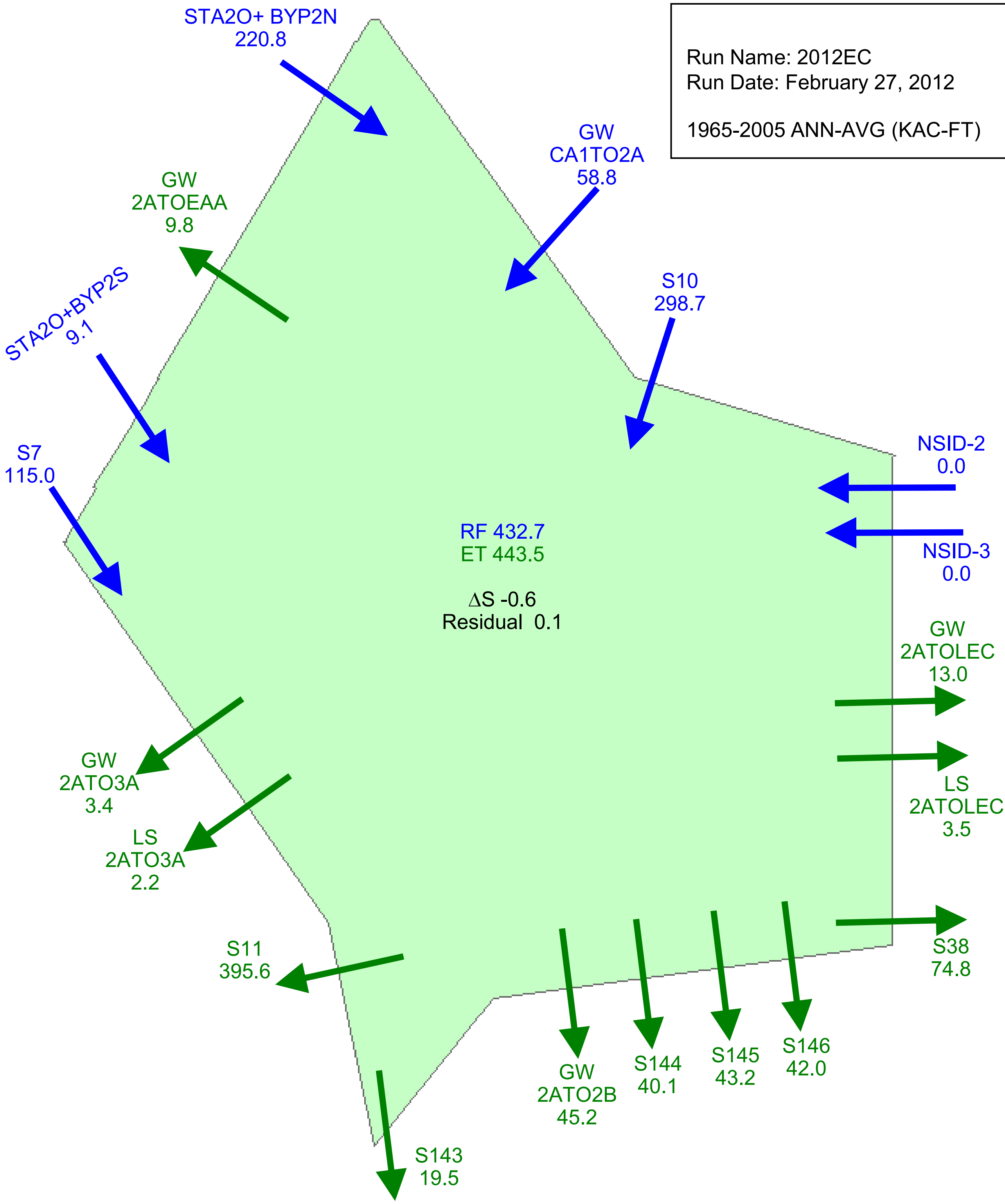
1965-2005 ANN-AVG (KAC-FT)



WATER CONSERVATION AREA-2A

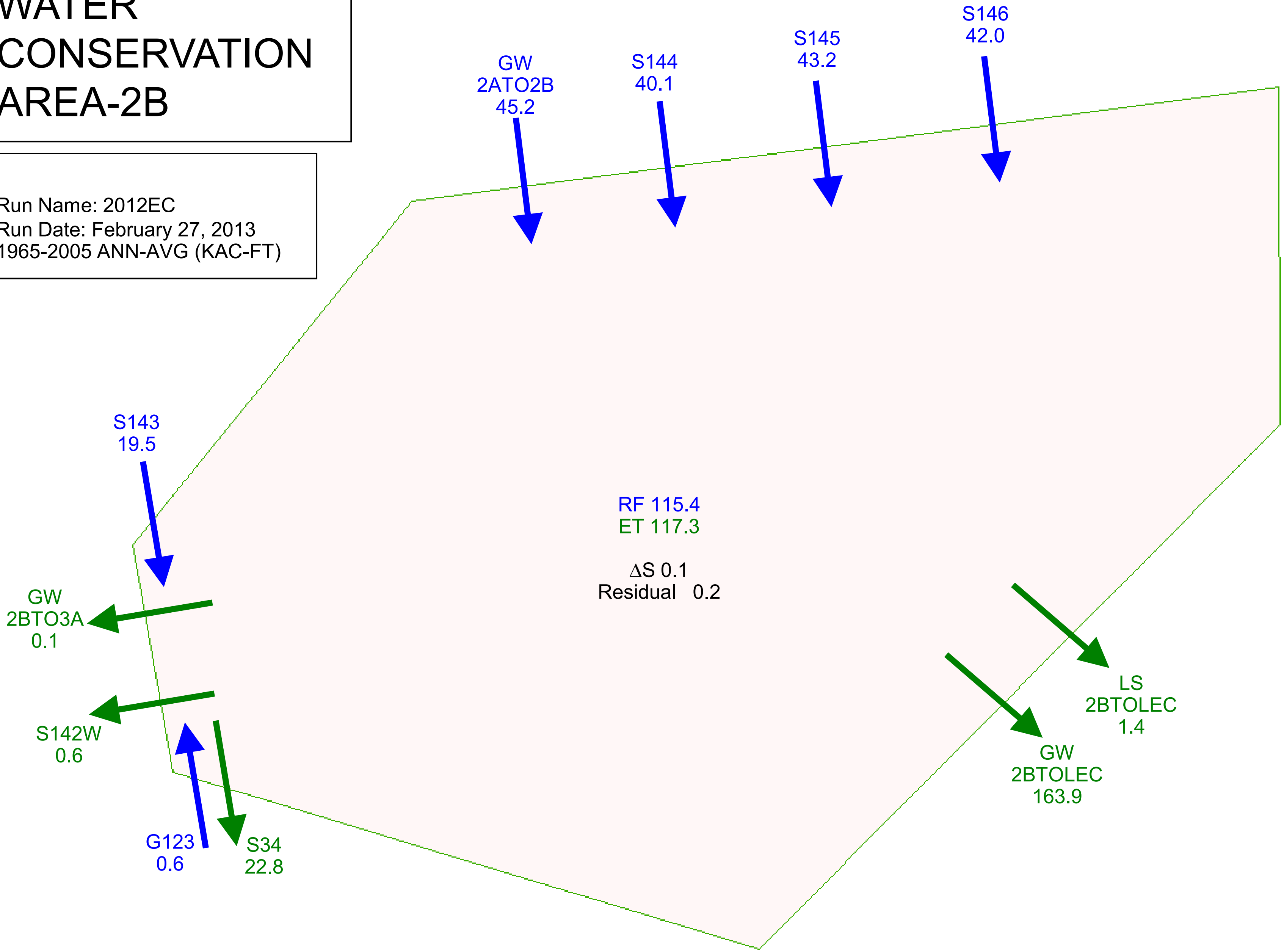
Run Name: 2012EC
Run Date: February 27, 2012

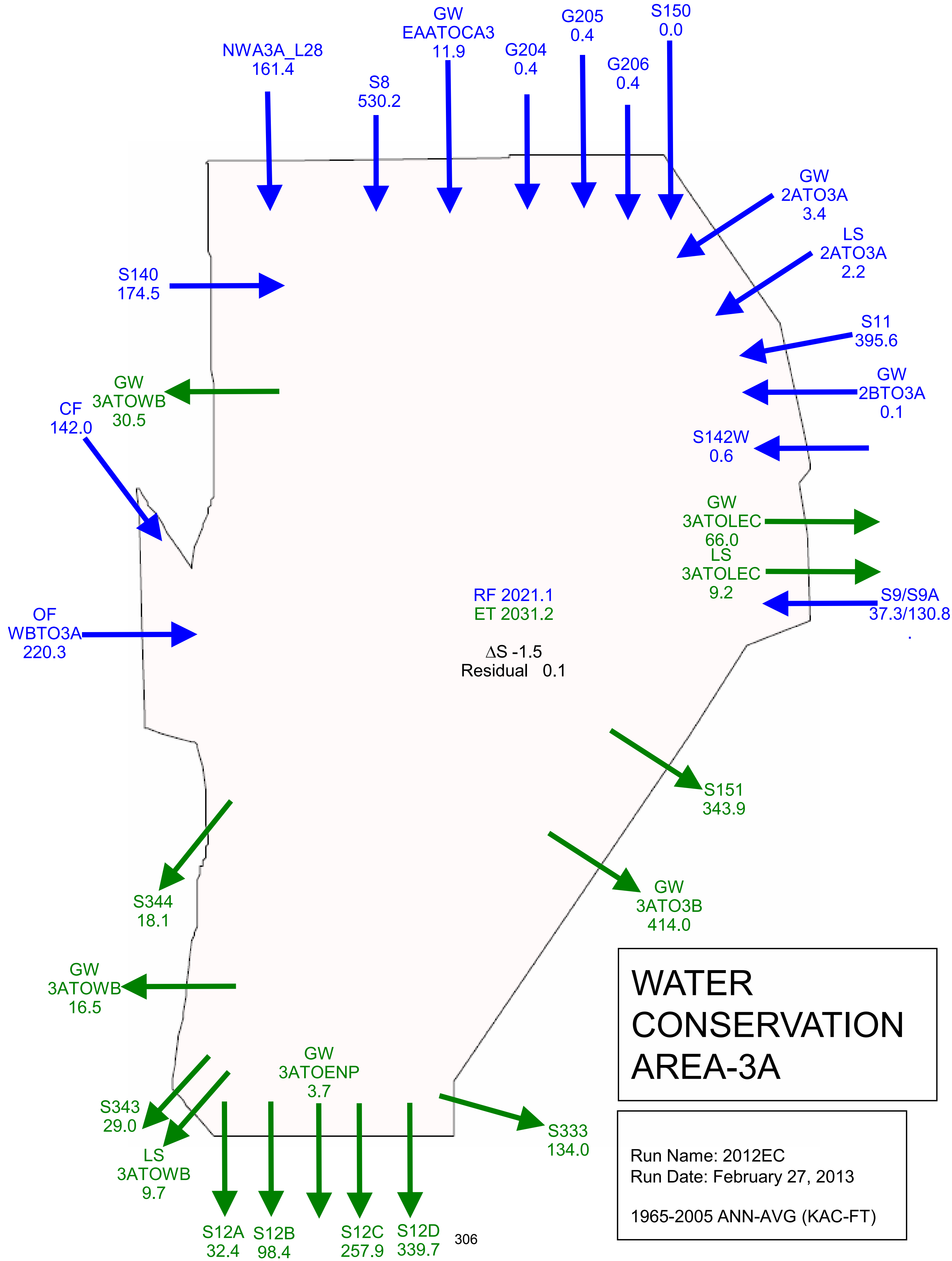
1965-2005 ANN-AVG (KAC-FT)



WATER CONSERVATION AREA-2B

Run Name: 2012EC
Run Date: February 27, 2013
1965-2005 ANN-AVG (KAC-FT)



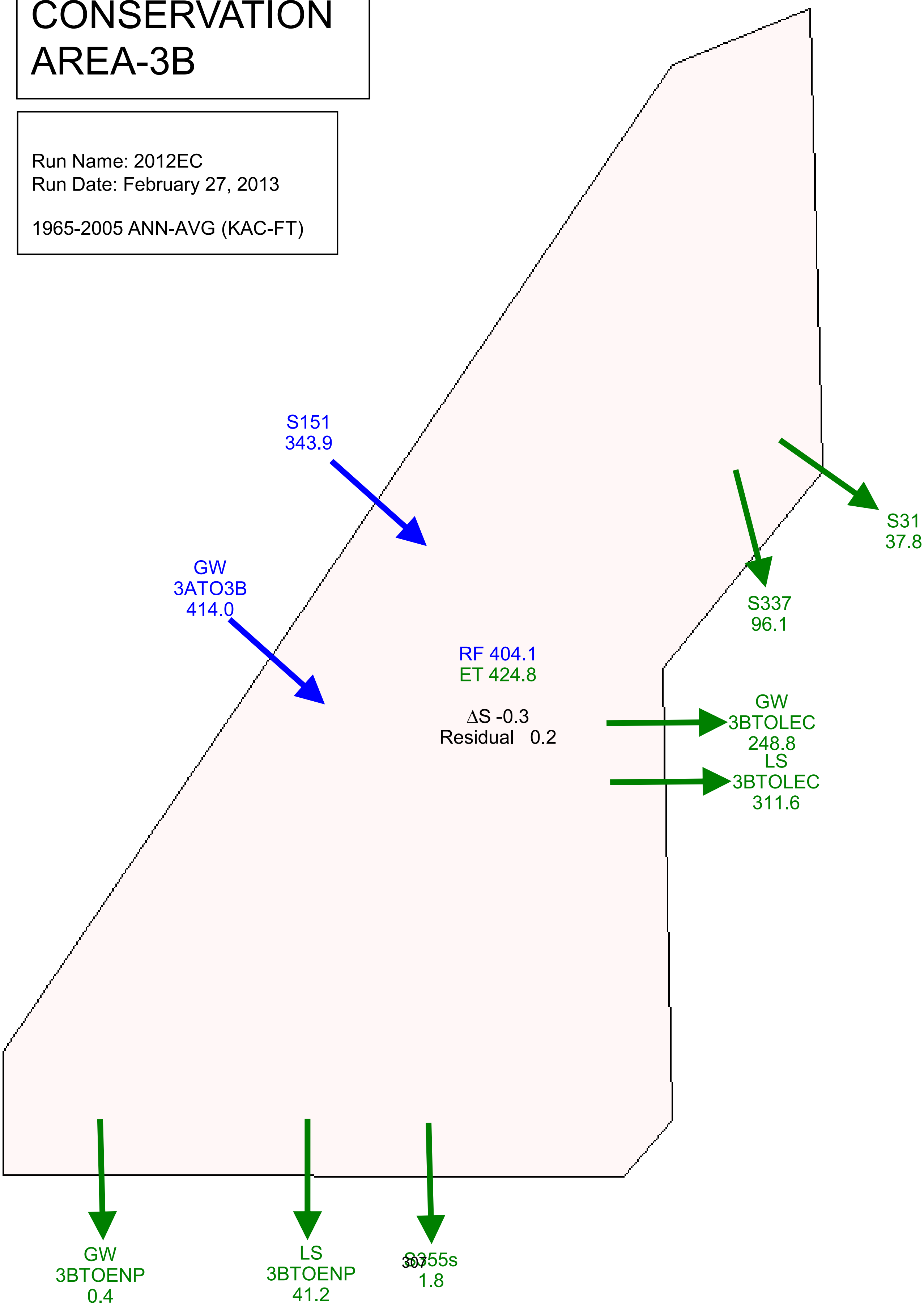


NWA3A_L28
161.4
S8
530.2
GW
EAATOCA3
11.9
G204
0.4
G205
0.4
G206
0.4
S150
0.0
GW
2ATO3A
3.4
LS
2ATO3A
2.2
S11
395.6
GW
2BTO3A
0.1
S142W
0.6
S140
174.5
GW
3ATOWB
30.5
CF
142.0
GW
3ATOLEC
66.0
LS
3ATOLEC
9.2
S9/S9A
37.3/130.8
OF
WBTO3A
220.3
RF 2021.1
ET 2031.2
ΔS -1.5
Residual 0.1
S151
343.9
GW
3ATO3B
414.0
S344
18.1
GW
3ATOWB
16.5
GW
3ATOENP
3.7
S343
29.0
LS
3ATOWB
9.7
S333
134.0
S12A
32.4
S12B
98.4
S12C
257.9
S12D
339.7
WATER
CONSERVATION
AREA-3A
Run Name: 2012EC
Run Date: February 27, 2013
1965-2005 ANN-AVG (KAC-FT)

WATER CONSERVATION AREA-3B

Run Name: 2012EC
Run Date: February 27, 2013

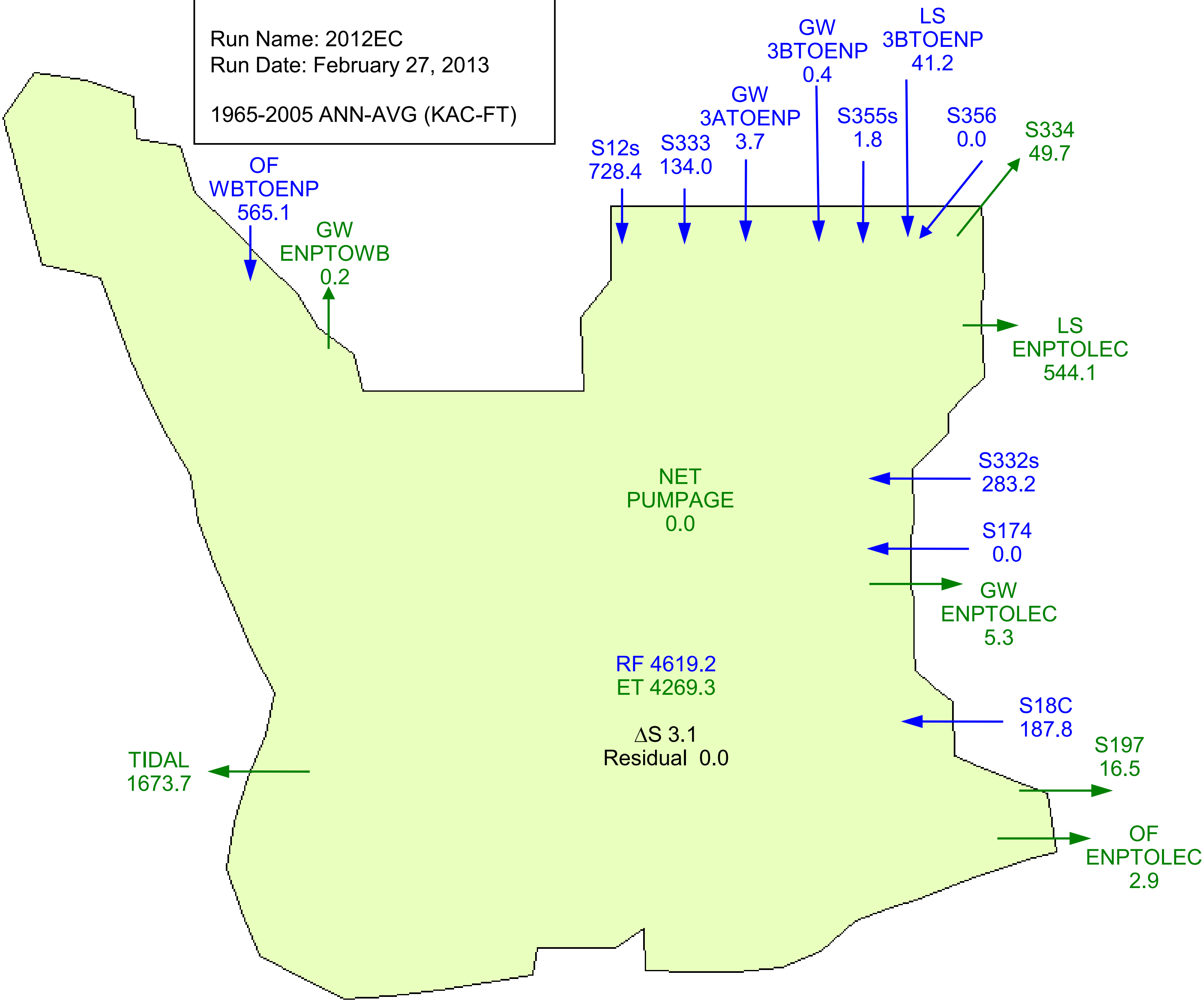
1965-2005 ANN-AVG (KAC-FT)



EVERGLADES NATIONAL PARK

Run Name: 2012EC
Run Date: February 27, 2013

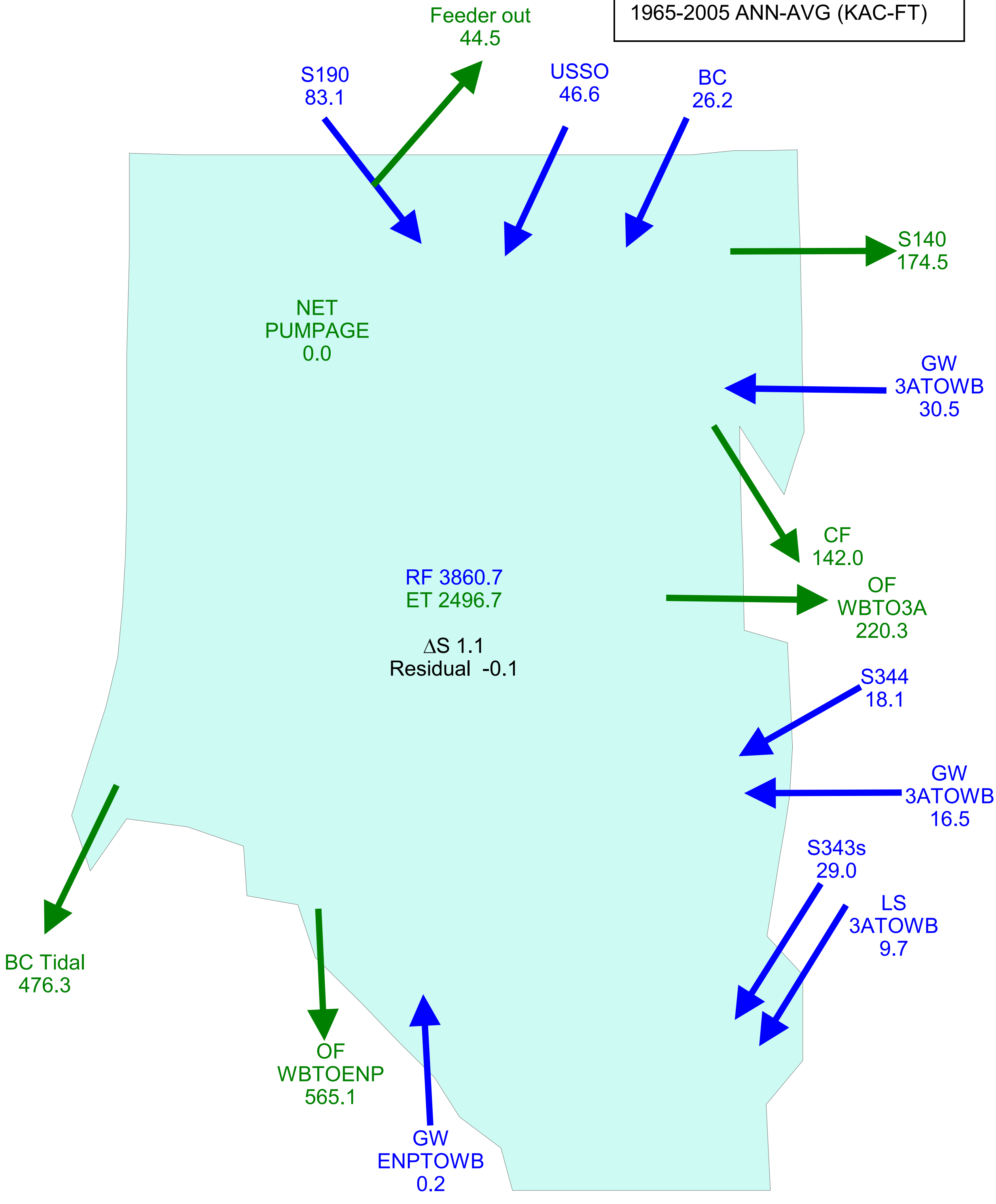
1965-2005 ANN-AVG (KAC-FT)



WESTERN BASINS

Run Name: 2012EC
Run Date: February 27, 2013

1965-2005 ANN-AVG (KAC-FT)

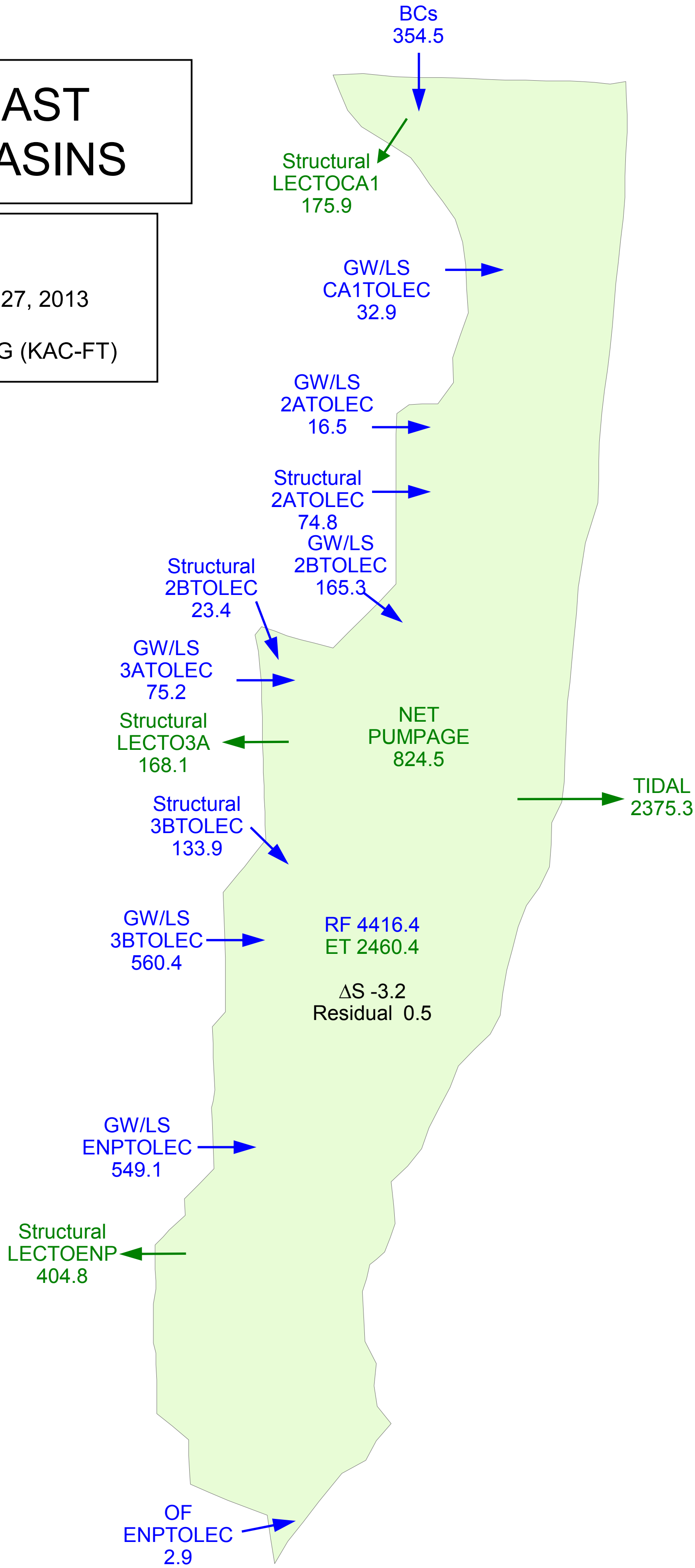


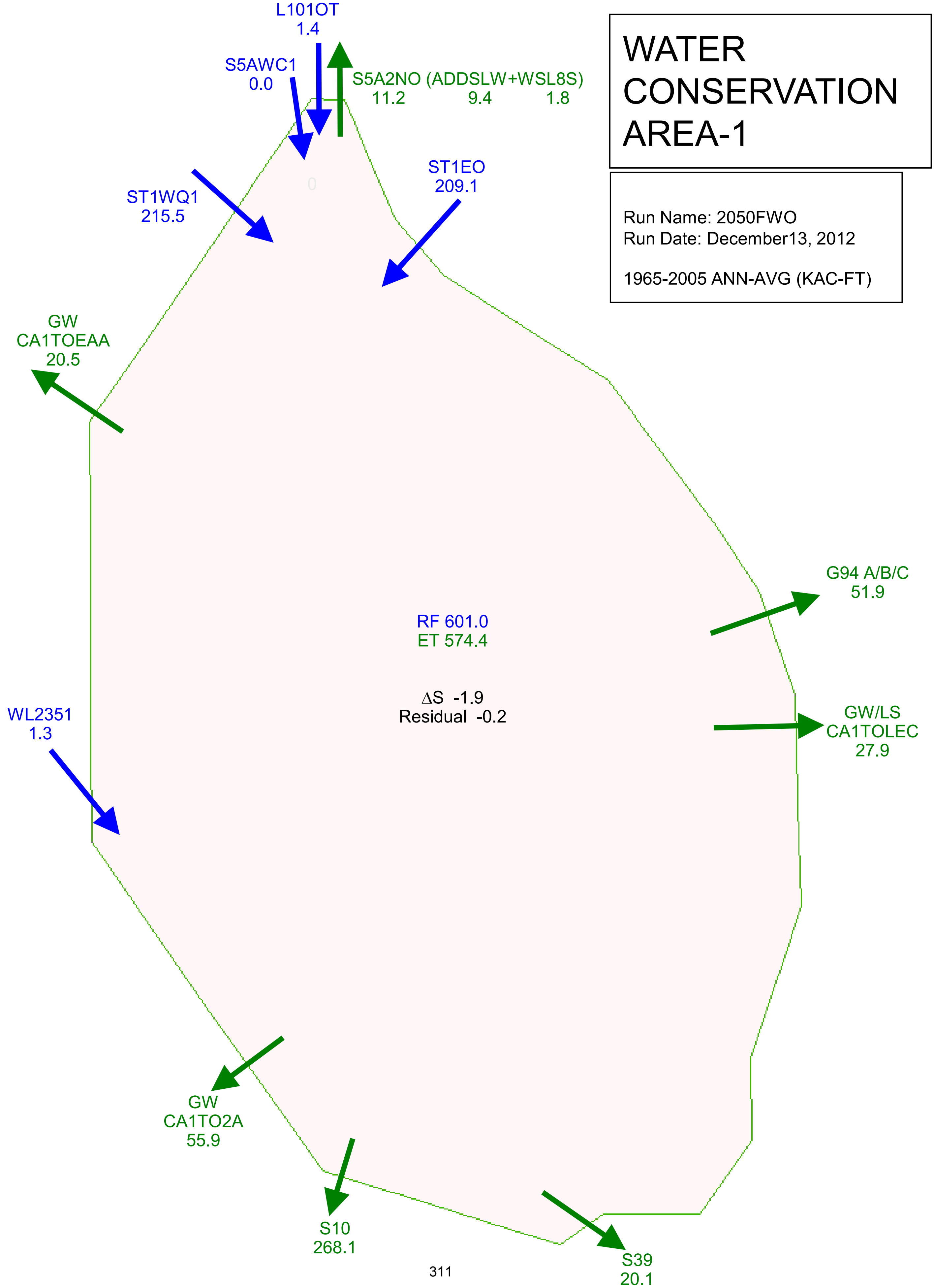
Note: Western Basins (WB) constitute L-28 Interceptor, Feeder Canal, L-28 Gap and East Collier Basins

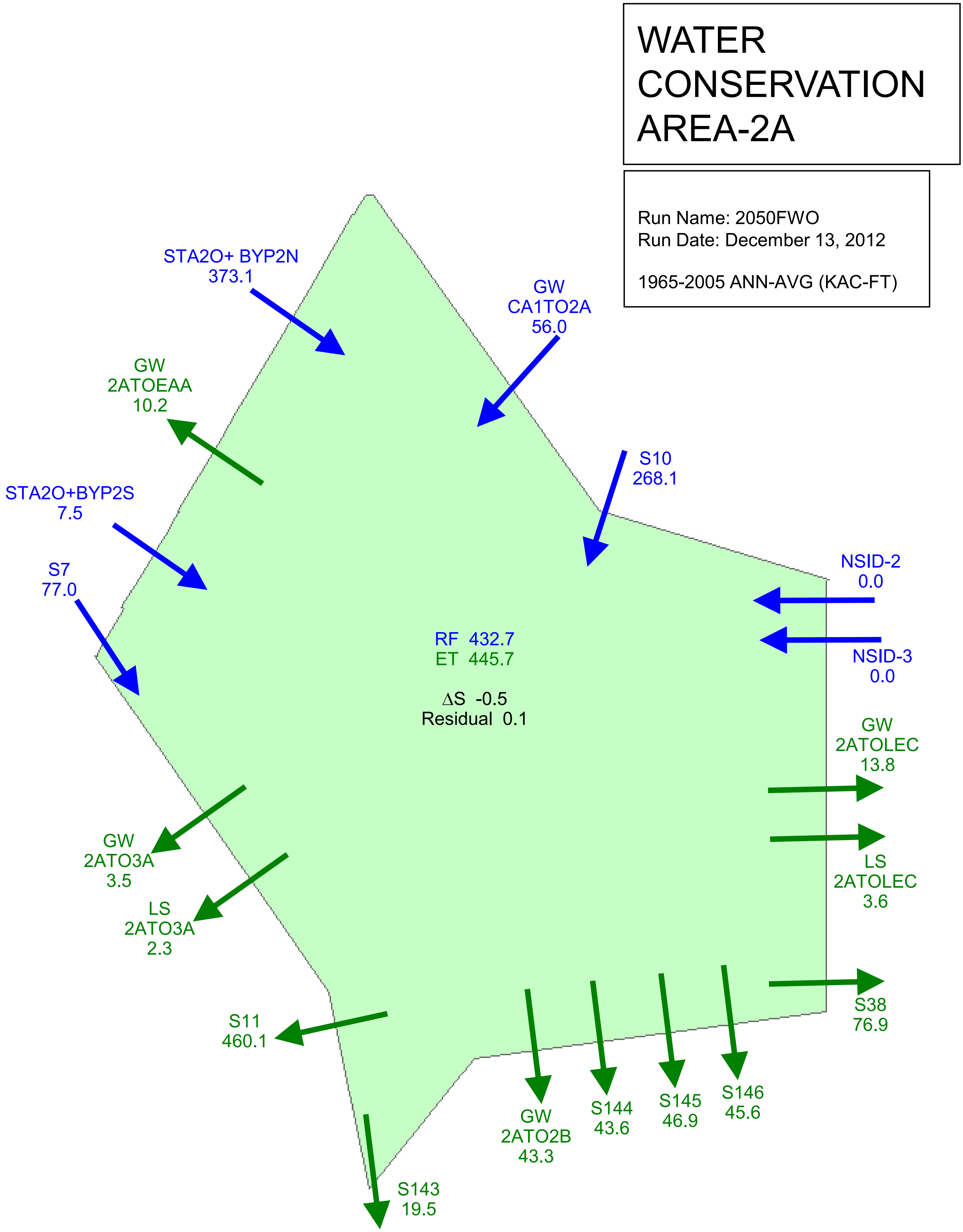
LOWER EAST COAST BASINS

Run Name: 2012EC
Run Date: February 27, 2013

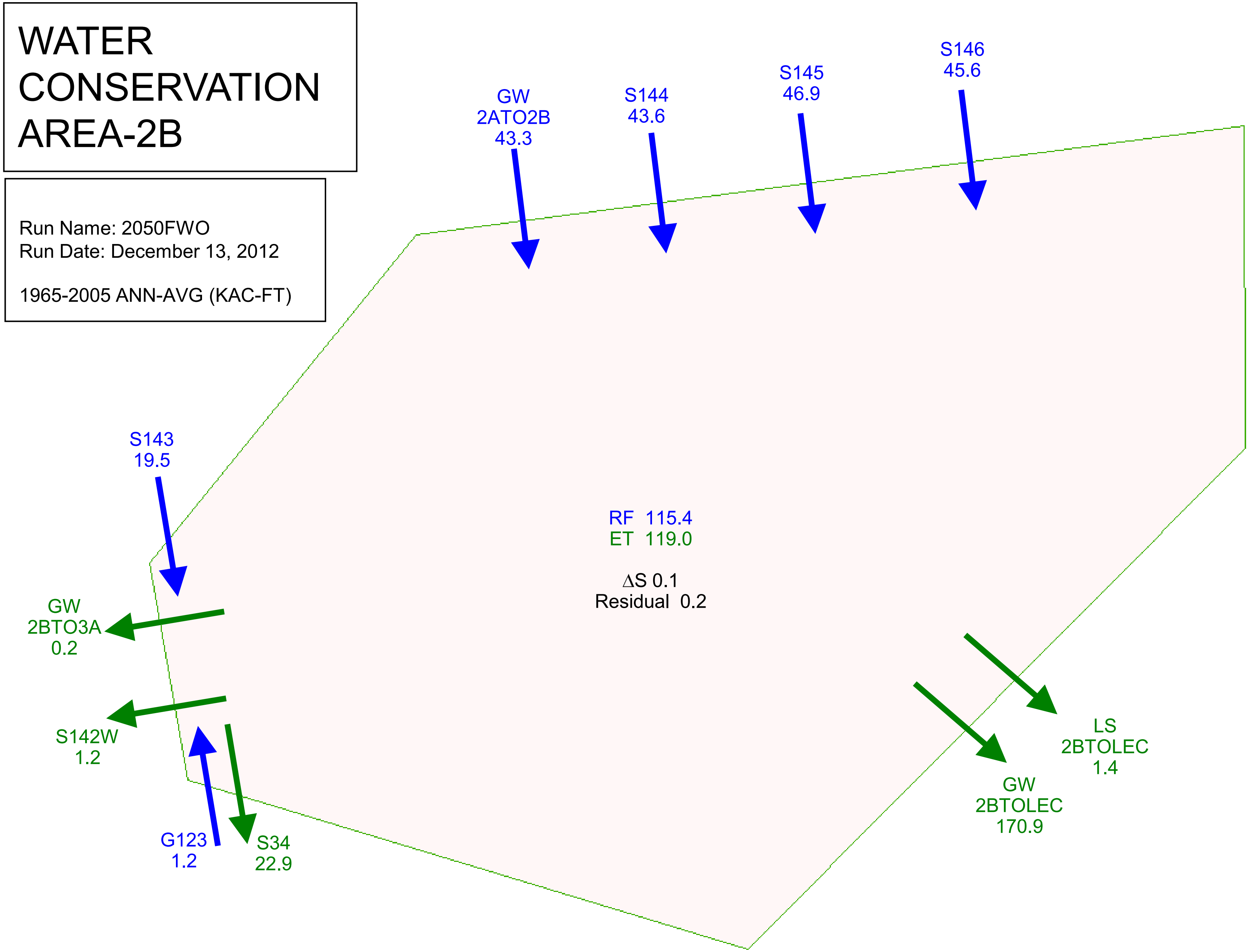
1965-2005 ANN-AVG (KAC-FT)

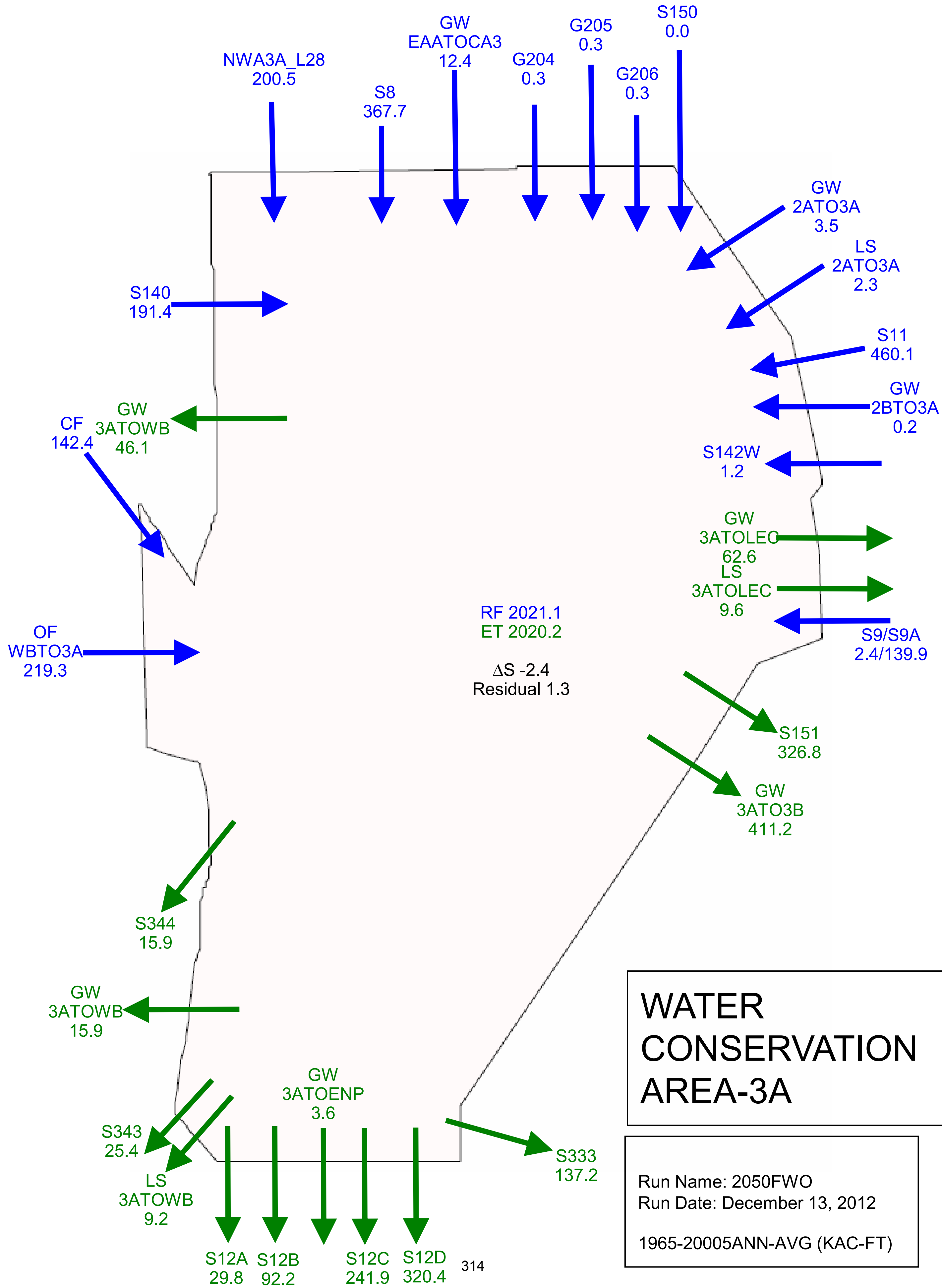



WATER CONSERVATION AREA-1
Run Name: 2050FWO
Run Date: December13, 2012
1965-2005 ANN-AVG (KAC-FT)
L101OT
1.4
S5AWC1
0.0
S5A2NO (ADDSLW+WSL8S)
11.2 9.4 1.8
ST1WQ1
215.5
ST1EO
209.1
GW
CA1TOEAA
20.5
G94 A/B/C
51.9
RF 601.0
ET 574.4
ΔS -1.9
Residual -0.2
WL2351
1.3
GW/LS
CA1TOLEC
27.9
GW
CA1TO2A
55.9
S10
268.1
S39
20.1



WATER CONSERVATION AREA-2A
Run Name: 2050FWO
Run Date: December 13, 2012
1965-2005 ANN-AVG (KAC-FT)
STA2O+ BYP2N 373.1
GW CA1TO2A 56.0
GW 2ATOEAA 10.2
S10 268.1
STA2O+BYP2S 7.5
NSID-2 0.0
S7 77.0
RF 432.7
ET 445.7
NSID-3 0.0
ΔS -0.5
Residual 0.1
GW 2ATOLEC 13.8
GW 2ATO3A 3.5
LS 2ATOLEC 3.6
LS 2ATO3A 2.3
S38 76.9
S11 460.1
S146 45.6
S145 46.9
S144 43.6
GW 2ATO2B 43.3
S143 19.5


WATER CONSERVATION AREA-2B
Run Name: 2050FWO
Run Date: December 13, 2012
1965-2005 ANN-AVG (KAC-FT)
GW 2ATO2B 43.3
S144 43.6
S145 46.9
S146 45.6
S143 19.5
RF 115.4
ET 119.0
ΔS 0.1
Residual 0.2
GW 2BTO3A 0.2
S142W 1.2
G123 1.2
S34 22.9
LS 2BTOLEC 1.4
GW 2BTOLEC 170.9



WATER CONSERVATION AREA-3A

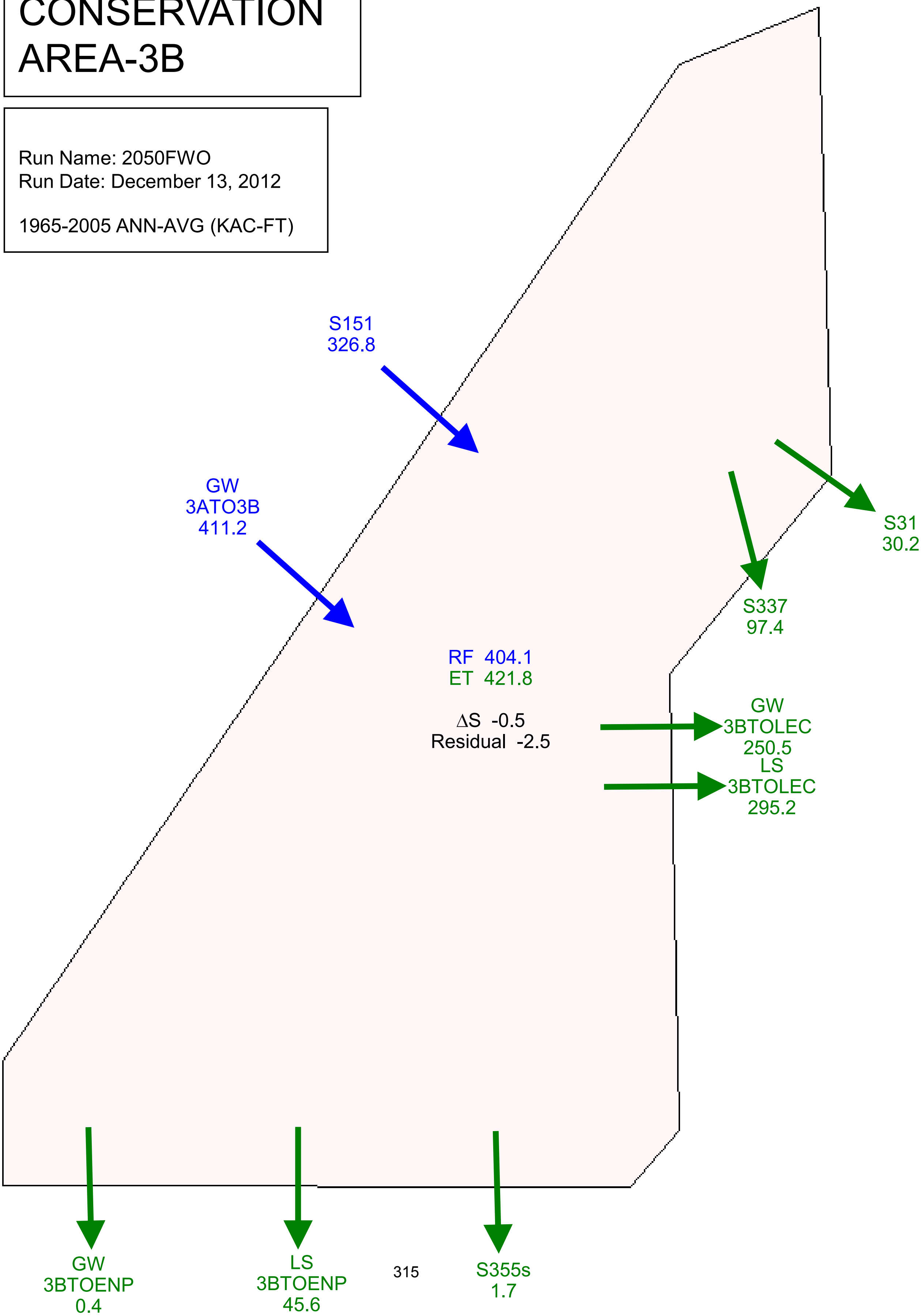
Run Name: 2050FWO
Run Date: December 13, 2012

1965-20005ANN-AVG (KAC-FT)

WATER CONSERVATION AREA-3B

Run Name: 2050FWO
Run Date: December 13, 2012

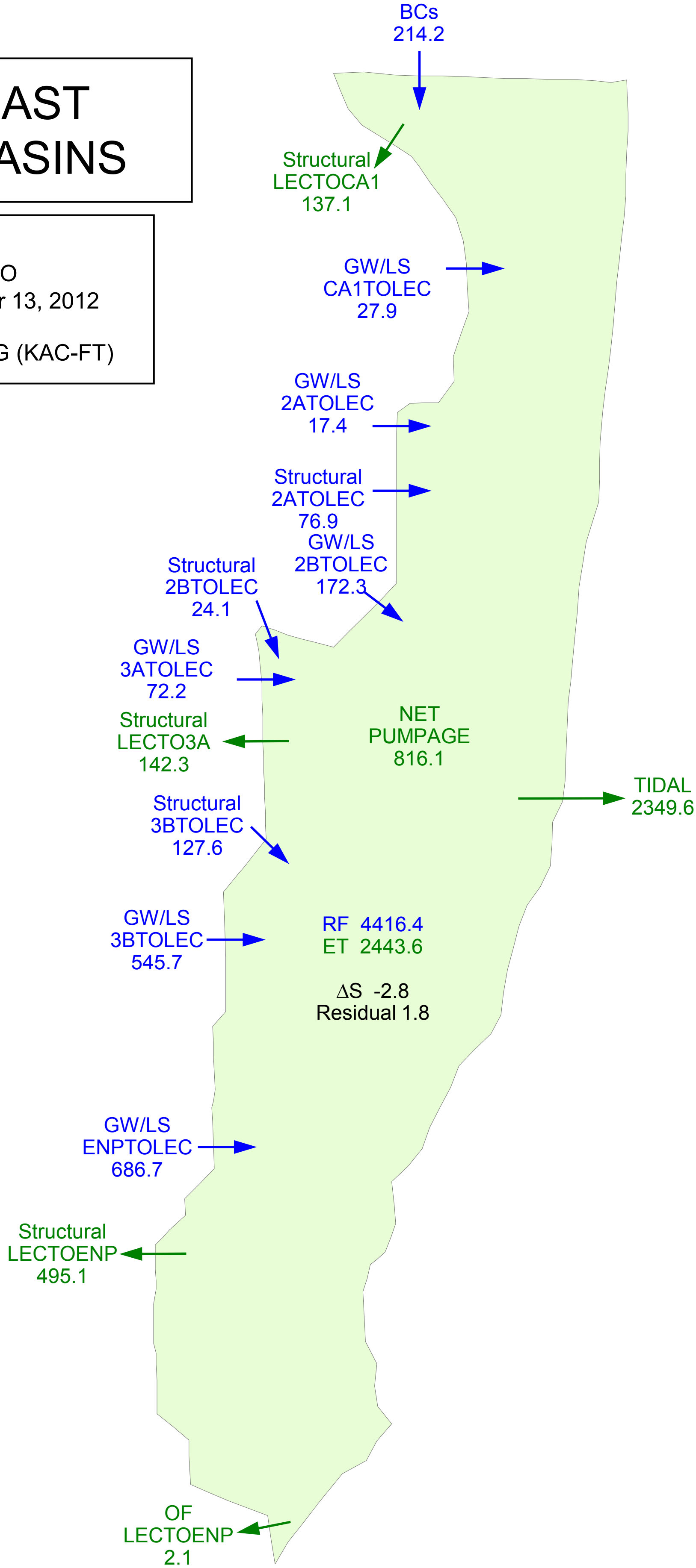
1965-2005 ANN-AVG (KAC-FT)

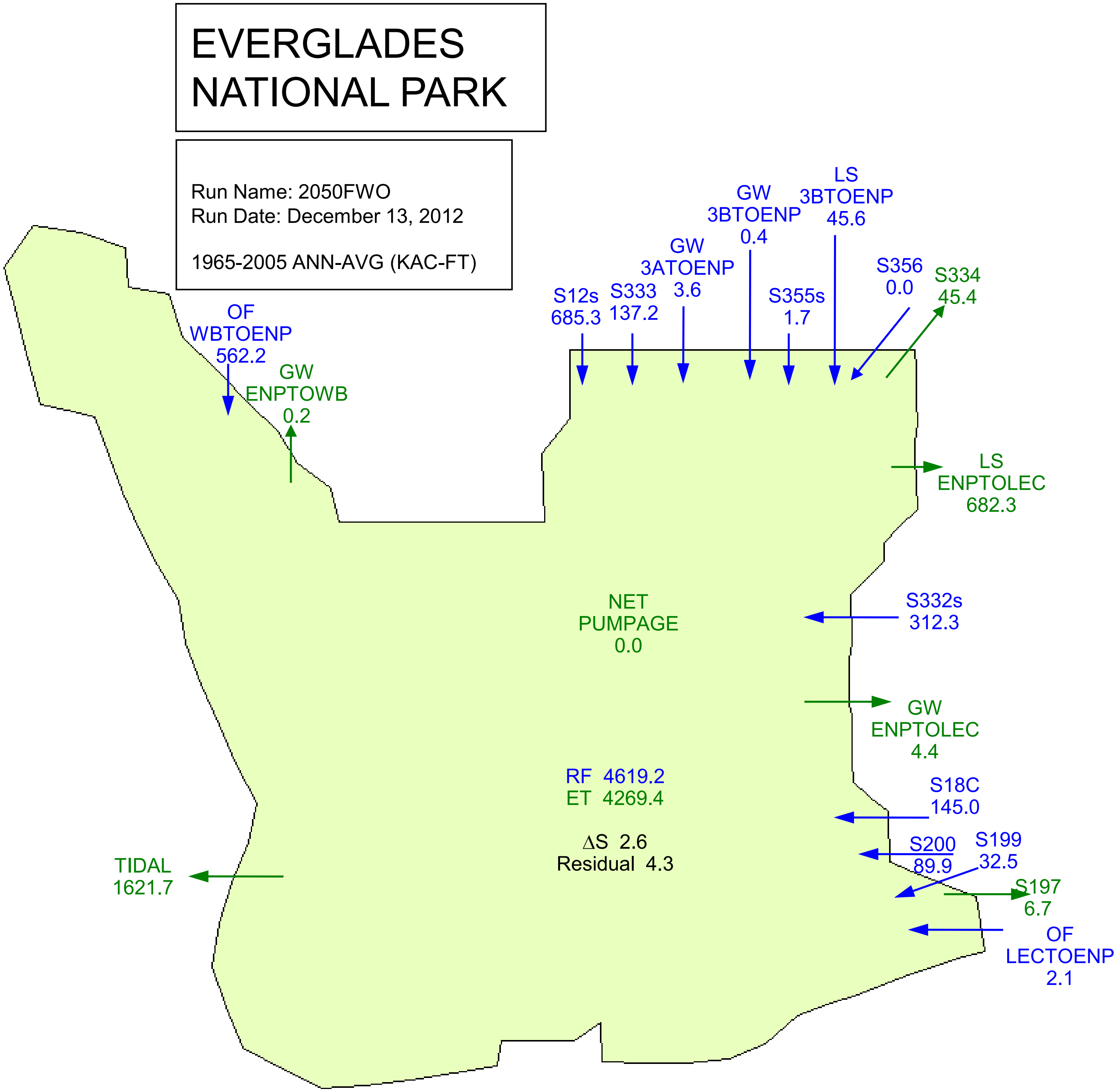


LOWER EAST COAST BASINS

Run Name: 2050FWO
Run Date: December 13, 2012

1965-2005 ANN-AVG (KAC-FT)



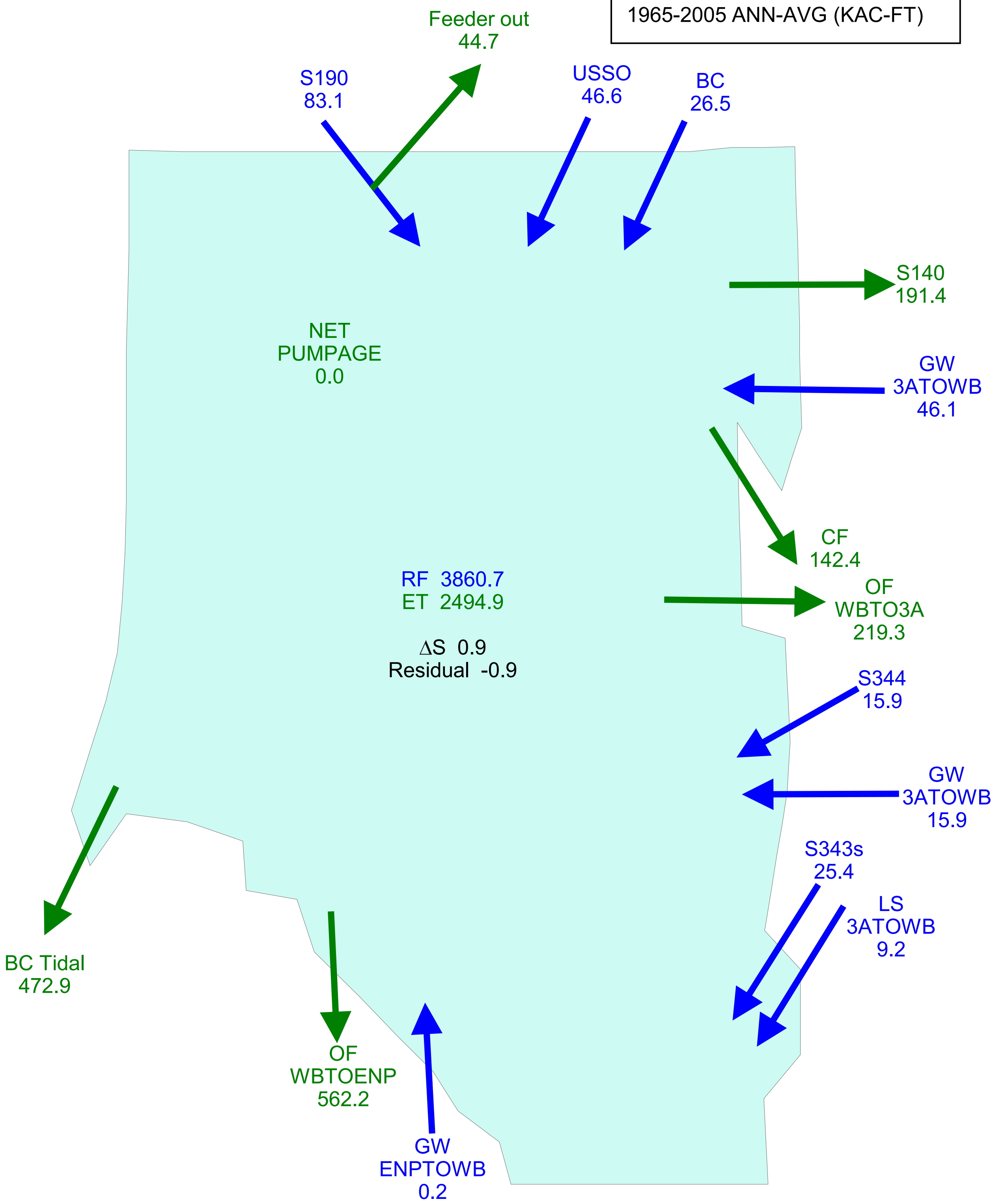


EVERGLADES
NATIONAL PARK
Run Name: 2050FWO
Run Date: December 13, 2012
1965-2005 ANN-AVG (KAC-FT)
OF
WBTOENP
562.2
GW
ENPTOWB
0.2
S12s
685.3
S333
137.2
GW
3ATOENP
3.6
GW
3BTOENP
0.4
S355s
1.7
LS
3BTOENP
45.6
S356
0.0
S334
45.4
LS
ENPTOLEC
682.3
NET
PUMPAGE
0.0
S332s
312.3
GW
ENPTOLEC
4.4
RF 4619.2
ET 4269.4
ΔS 2.6
Residual 4.3
S18C
145.0
S200
89.9
S199
32.5
S197
6.7
OF
LECTOENP
2.1
TIDAL
1621.7

WESTERN BASINS

Run Name: 2050
Run Date: December 13, 2012

1965-2005 ANN-AVG (KAC-FT)

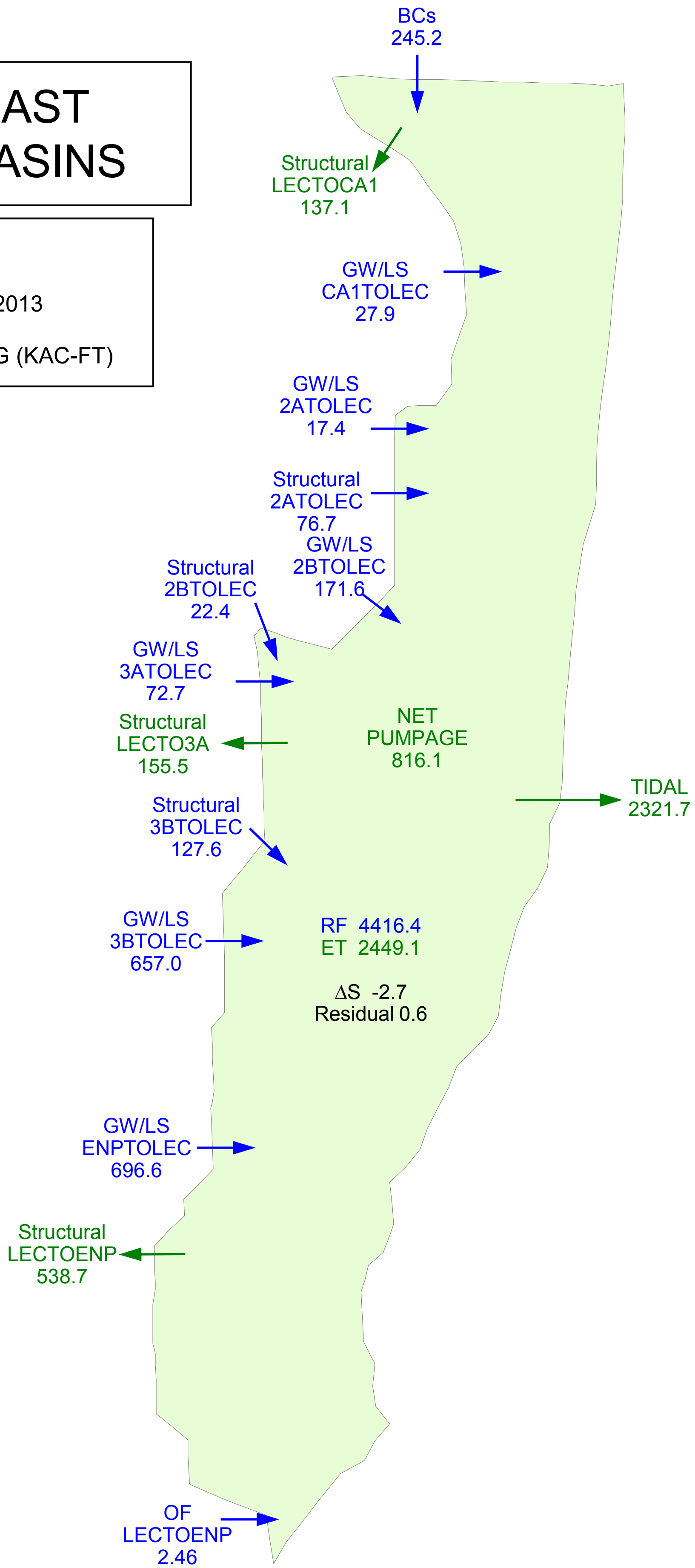


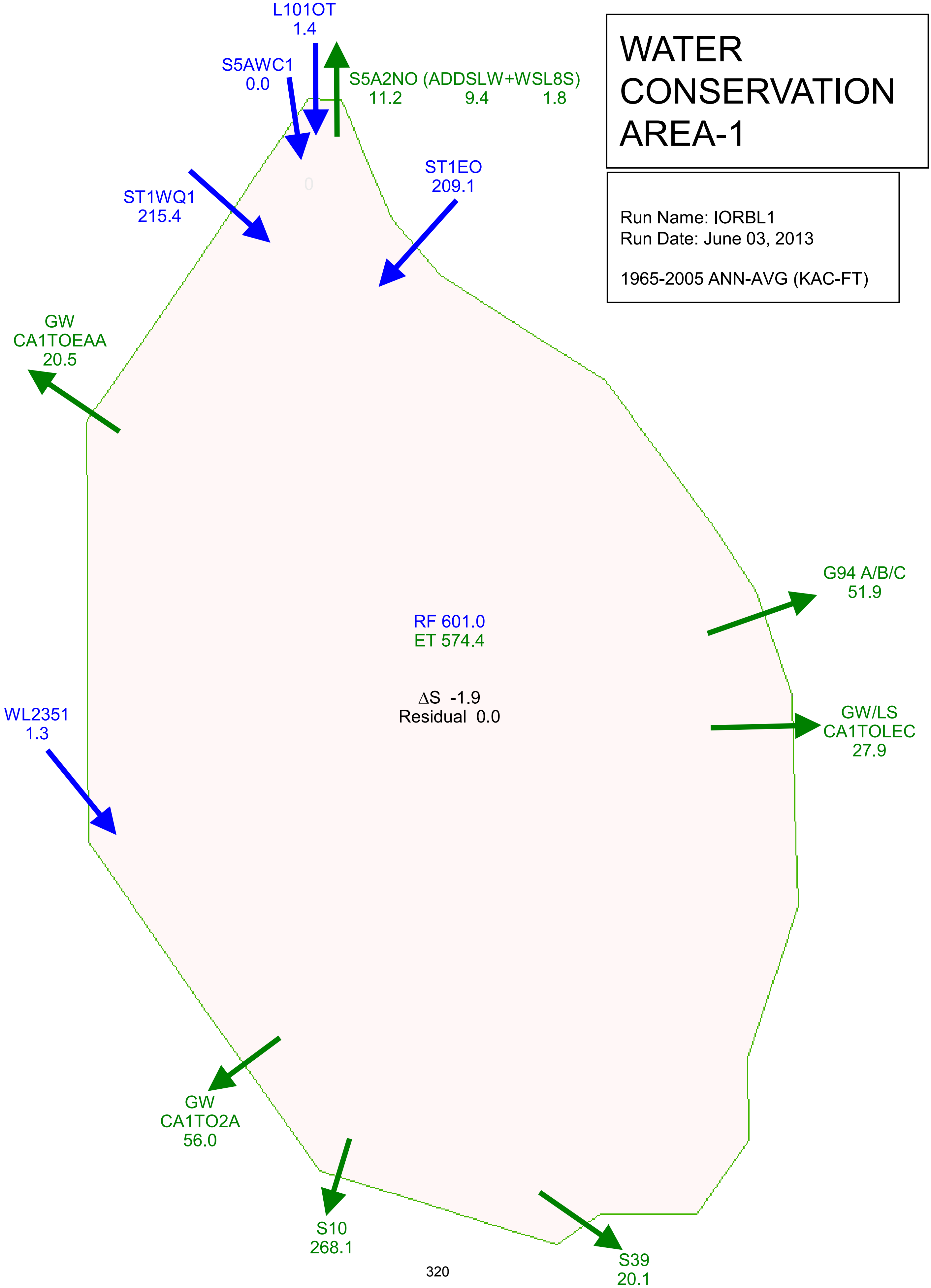
Note: Western Basins (WB) constitute L-28 Interceptor, Feeder Canal, L-28 Gap and East Collier Basins

LOWER EAST COAST BASINS

Run Name: IORBL1
Run Date: June 03, 2013

1965-2005 ANN-AVG (KAC-FT)

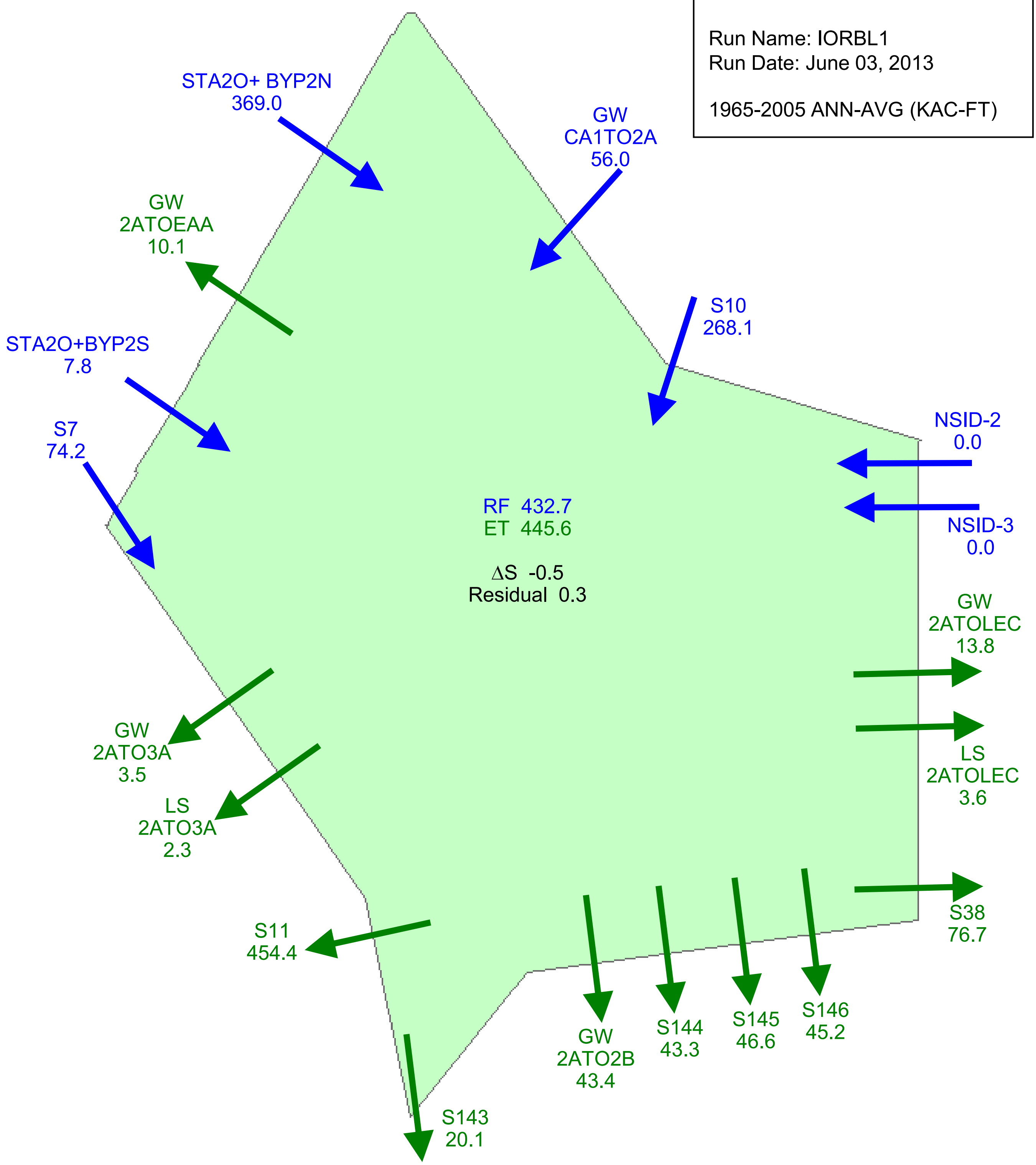


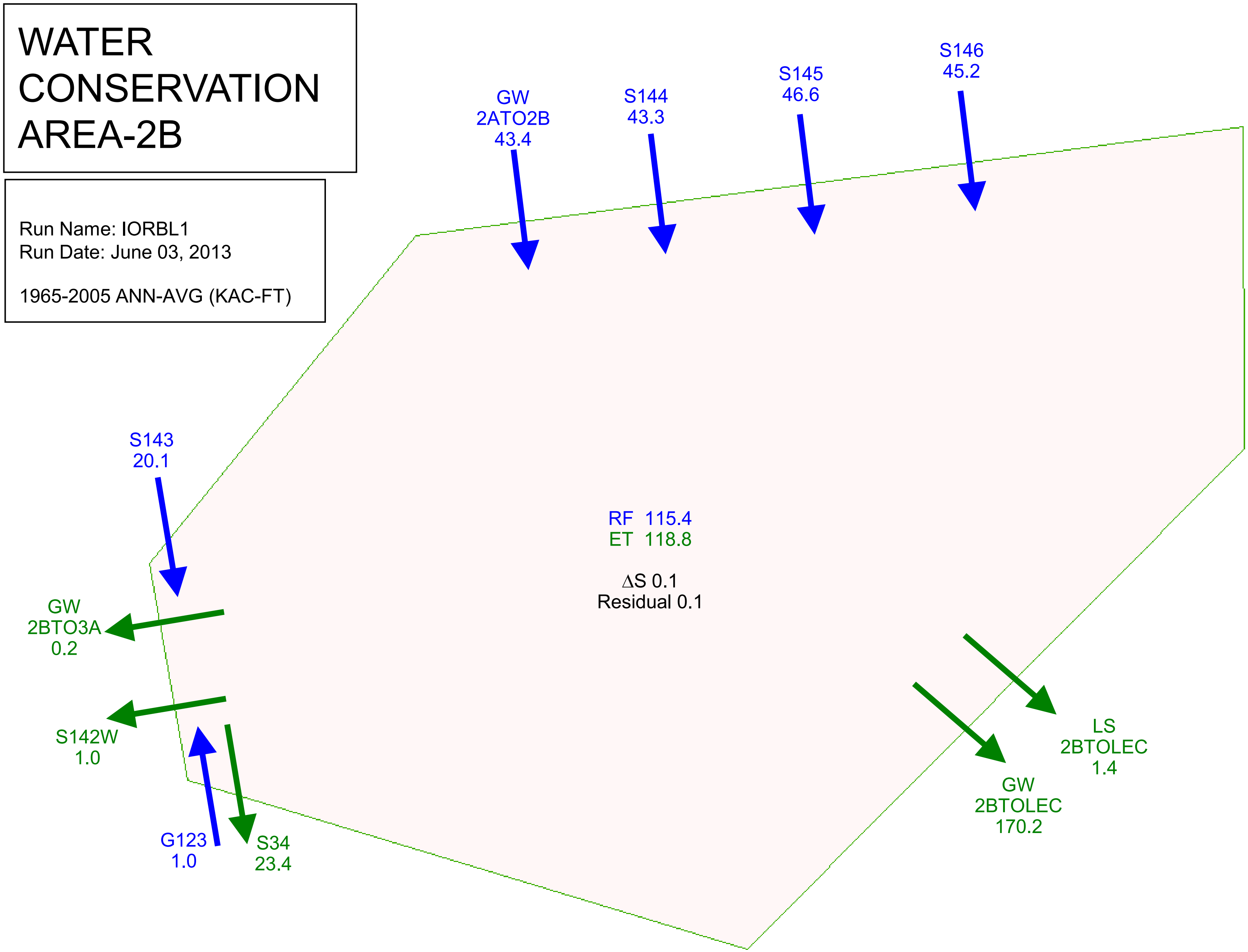

WATER CONSERVATION AREA-1
Run Name: IORBL1
Run Date: June 03, 2013
1965-2005 ANN-AVG (KAC-FT)
L101OT
1.4
S5AWC1
0.0
S5A2NO (ADDSLW+WSL8S)
11.2 9.4 1.8
ST1EO
209.1
ST1WQ1
215.4
GW
CA1TOEAA
20.5
G94 A/B/C
51.9
RF 601.0
ET 574.4
ΔS -1.9
Residual 0.0
WL2351
1.3
GW/LS
CA1TOLEC
27.9
GW
CA1TO2A
56.0
S10
268.1
S39
20.1

WATER CONSERVATION AREA-2A

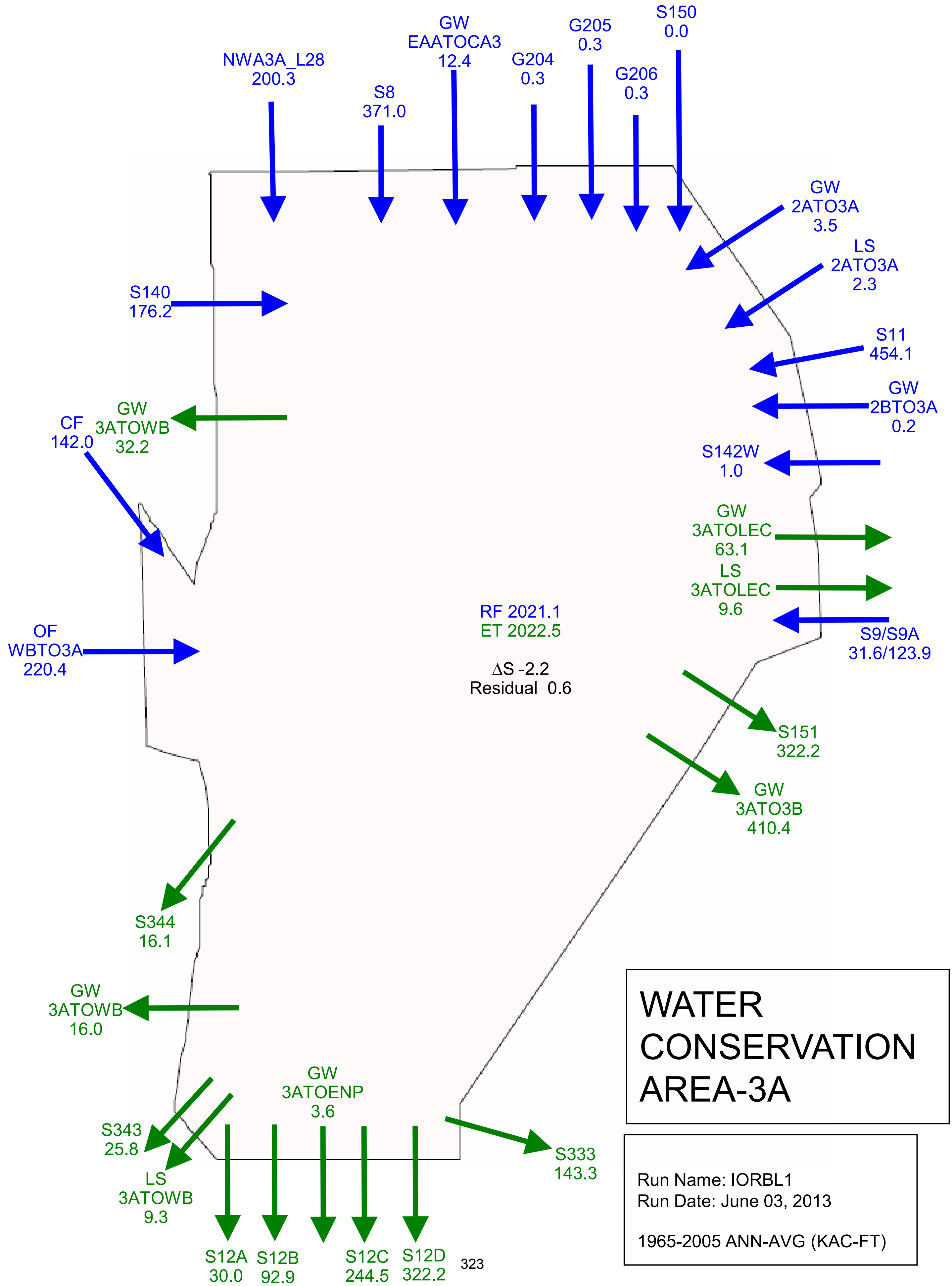
Run Name: IORBL1
Run Date: June 03, 2013

1965-2005 ANN-AVG (KAC-FT)





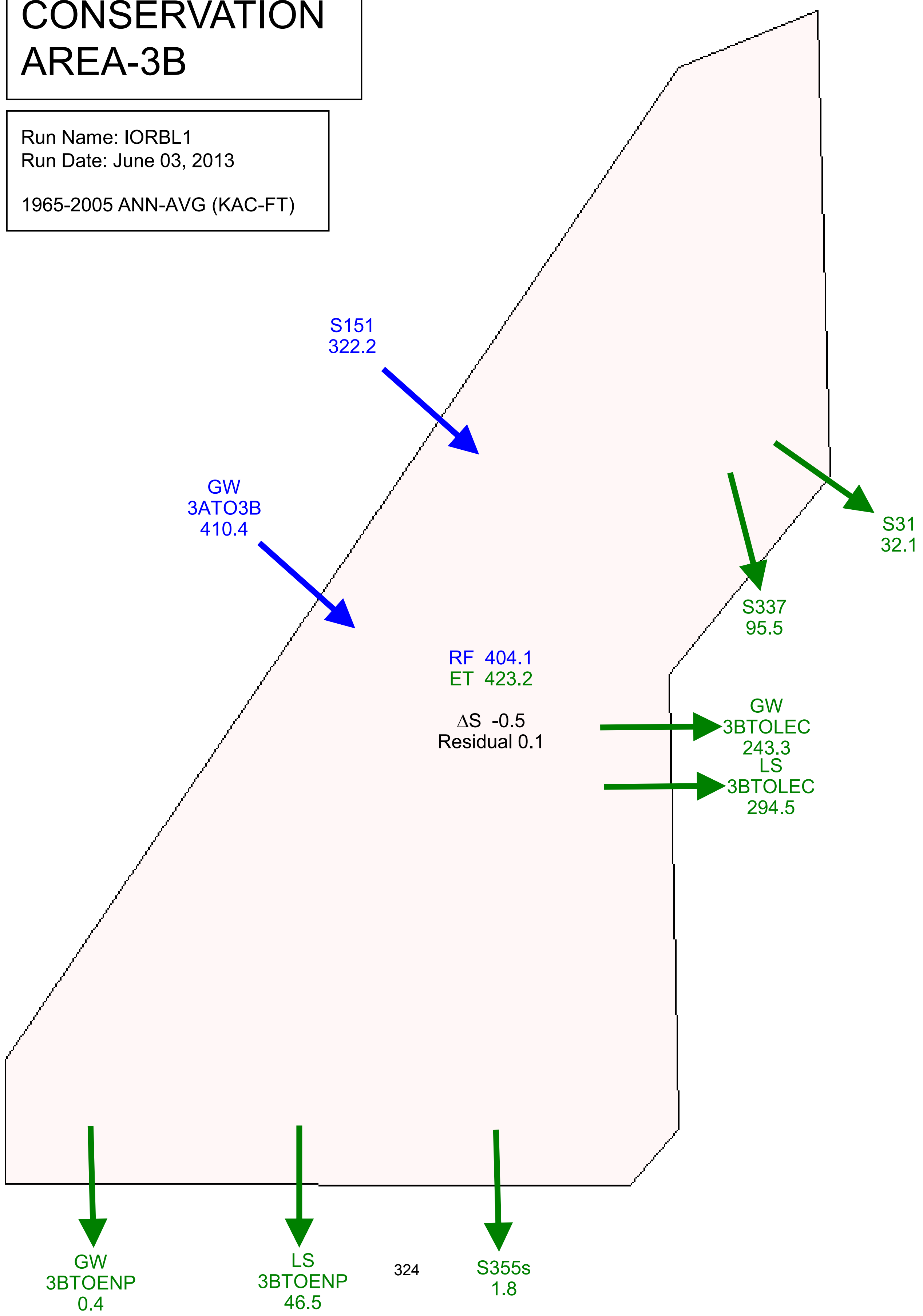
WATER CONSERVATION AREA-2B
Run Name: IORBL1
Run Date: June 03, 2013
1965-2005 ANN-AVG (KAC-FT)
GW 2ATO2B 43.4
S144 43.3
S145 46.6
S146 45.2
S143 20.1
RF 115.4
ET 118.8
ΔS 0.1
Residual 0.1
GW 2BTO3A 0.2
S142W 1.0
G123 1.0
S34 23.4
LS 2BTOLEC 1.4
GW 2BTOLEC 170.2


NWA3A_L28
200.3
S8
371.0
GW
EAATOCA3
12.4
G204
0.3
G205
0.3
G206
0.3
S150
0.0
GW
2ATO3A
3.5
LS
2ATO3A
2.3
S11
454.1
GW
2BTO3A
0.2
S142W
1.0
GW
3ATOLEC
63.1
LS
3ATOLEC
9.6
S9/S9A
31.6/123.9
S140
176.2
CF
142.0
GW
3ATOWB
32.2
OF
WBTO3A
220.4
RF 2021.1
ET 2022.5
ΔS -2.2
Residual 0.6
S151
322.2
GW
3ATO3B
410.4
S344
16.1
GW
3ATOWB
16.0
S343
25.8
LS
3ATOWB
9.3
GW
3ATOENP
3.6
S333
143.3
S12A
30.0
S12B
92.9
S12C
244.5
S12D
322.2
WATER
CONSERVATION
AREA-3A
Run Name: IORBL1
Run Date: June 03, 2013
1965-2005 ANN-AVG (KAC-FT)

WATER CONSERVATION AREA-3B

Run Name: IORBL1
Run Date: June 03, 2013

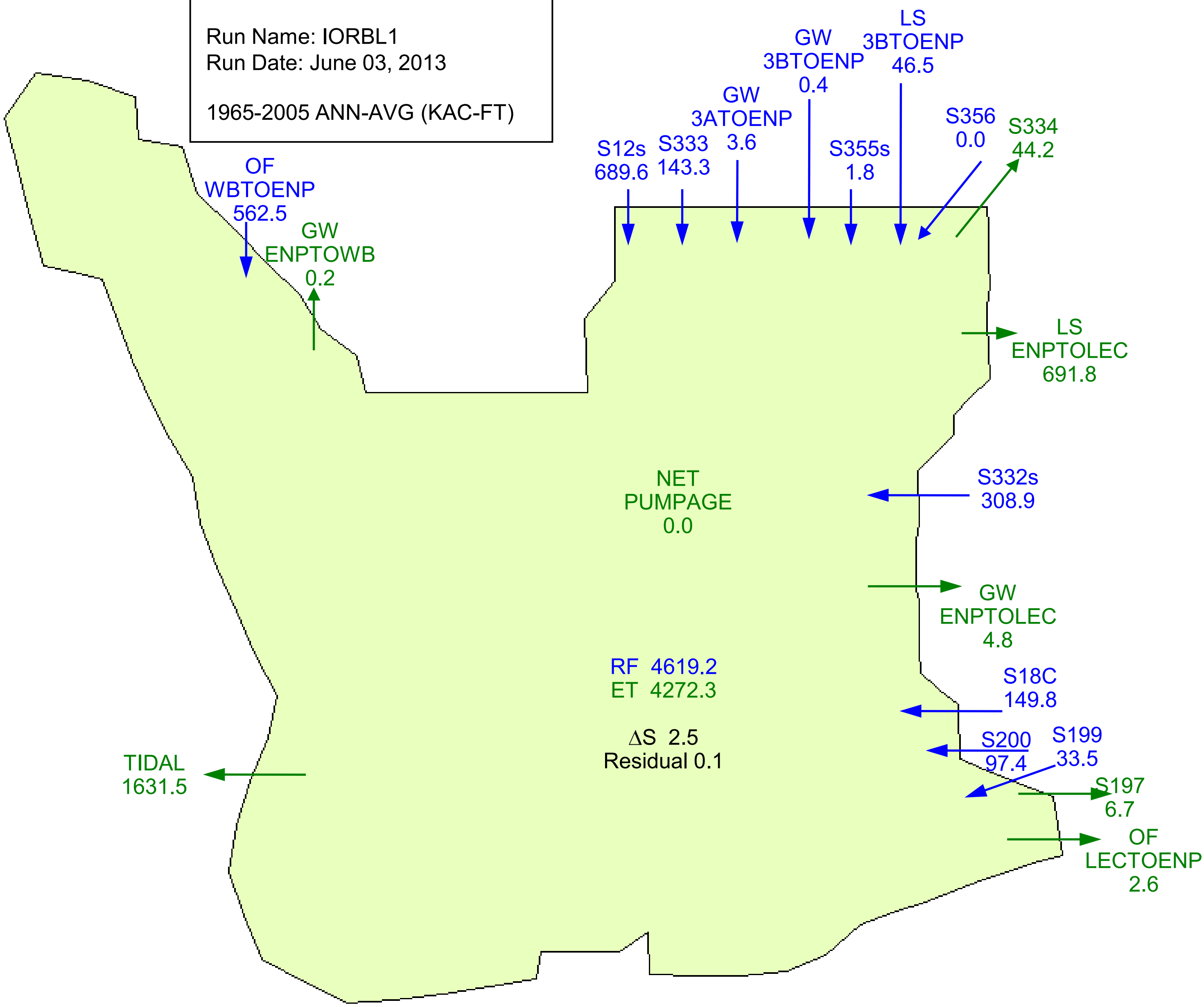
1965-2005 ANN-AVG (KAC-FT)



EVERGLADES NATIONAL PARK

Run Name: IORBL1
Run Date: June 03, 2013

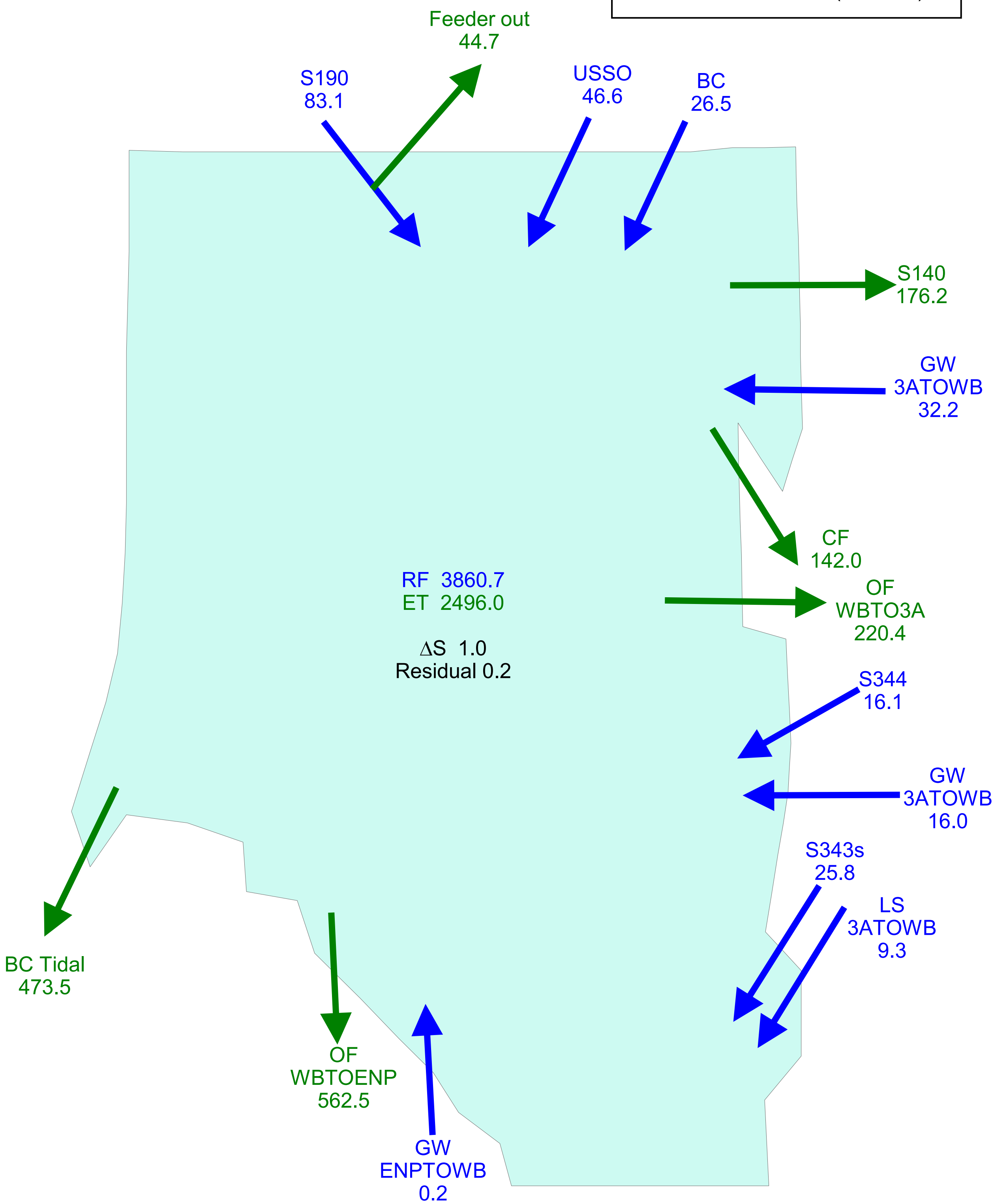
1965-2005 ANN-AVG (KAC-FT)



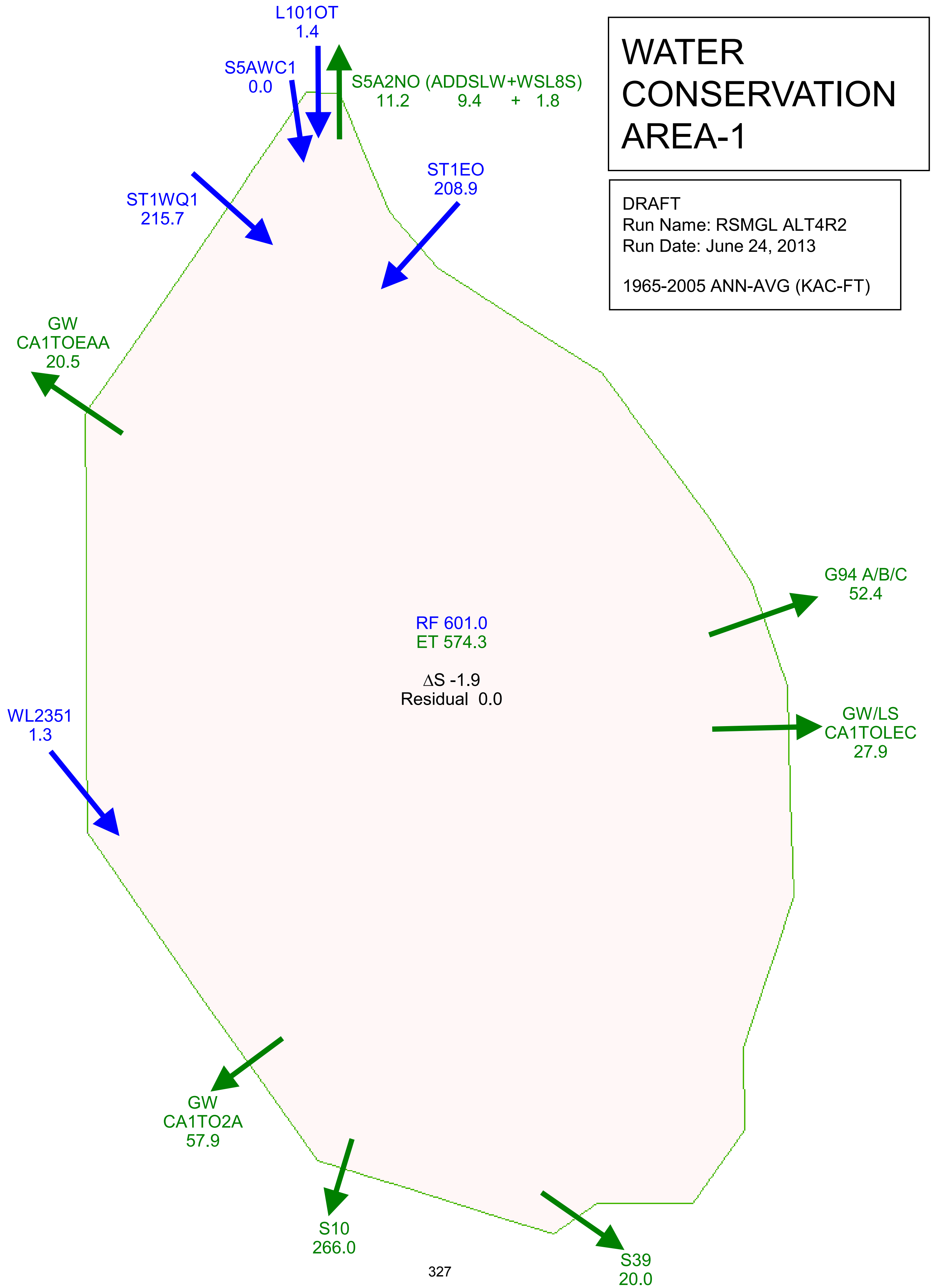
WESTERN BASINS

Run Name: IORBL1
Run Date: June 03, 2013

1965-2005 ANN-AVG (KAC-FT)



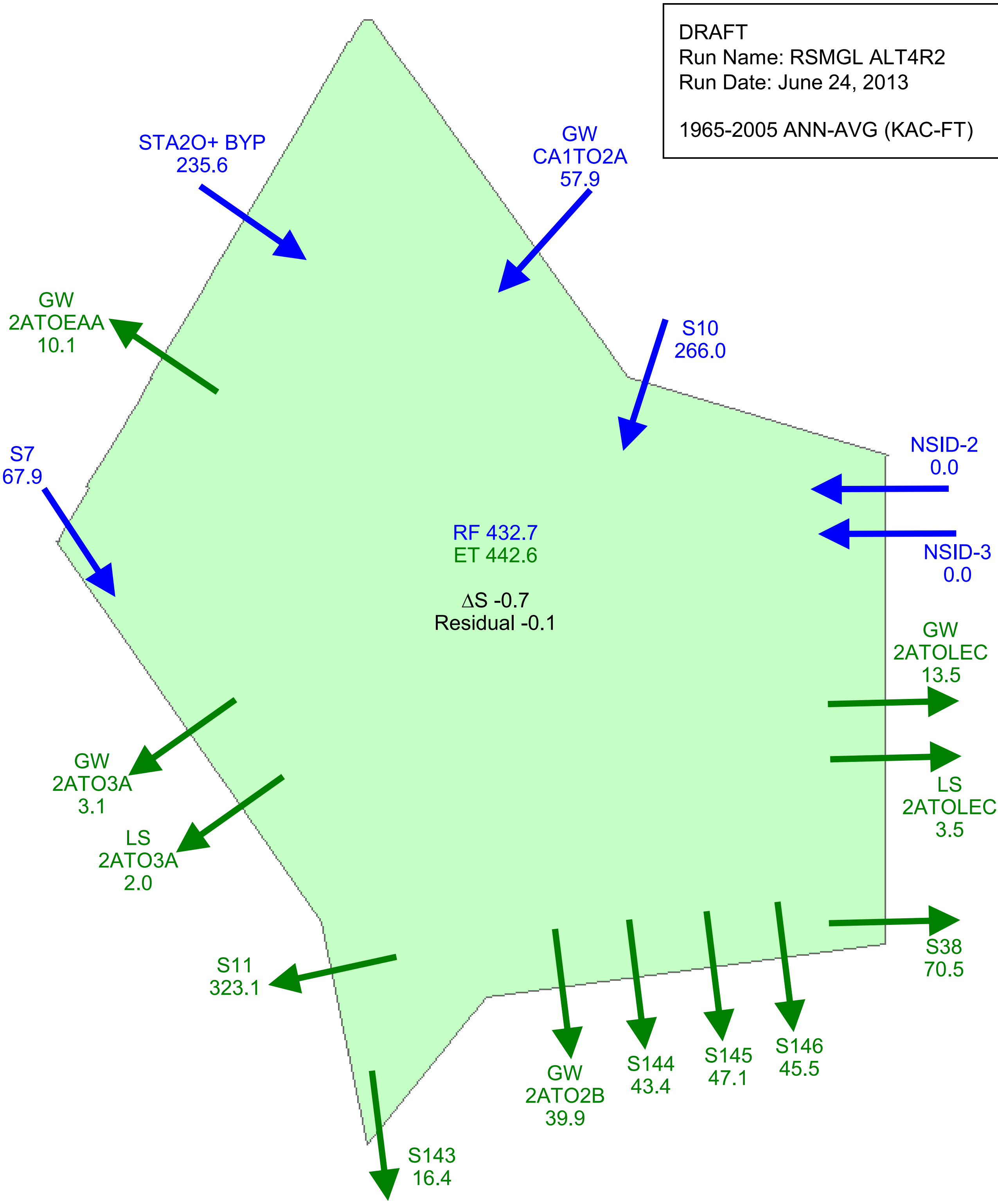
Note: Western Basins (WB) constitute L-28 Interceptor, Feeder Canal, L-28 Gap and East Collier Basins


WATER CONSERVATION AREA-1
DRAFT
Run Name: RSMGL ALT4R2
Run Date: June 24, 2013
1965-2005 ANN-AVG (KAC-FT)
L101OT
1.4
S5AWC1
0.0
S5A2NO (ADDSLW+WSL8S)
11.2 9.4 + 1.8
ST1WQ1
215.7
ST1EO
208.9
GW
CA1TOEAA
20.5
G94 A/B/C
52.4
RF 601.0
ET 574.3
ΔS -1.9
Residual 0.0
WL2351
1.3
GW/LS
CA1TOLEC
27.9
GW
CA1TO2A
57.9
S10
266.0
S39
20.0

WATER CONSERVATION AREA-2A

DRAFT
Run Name: RSMGL ALT4R2
Run Date: June 24, 2013

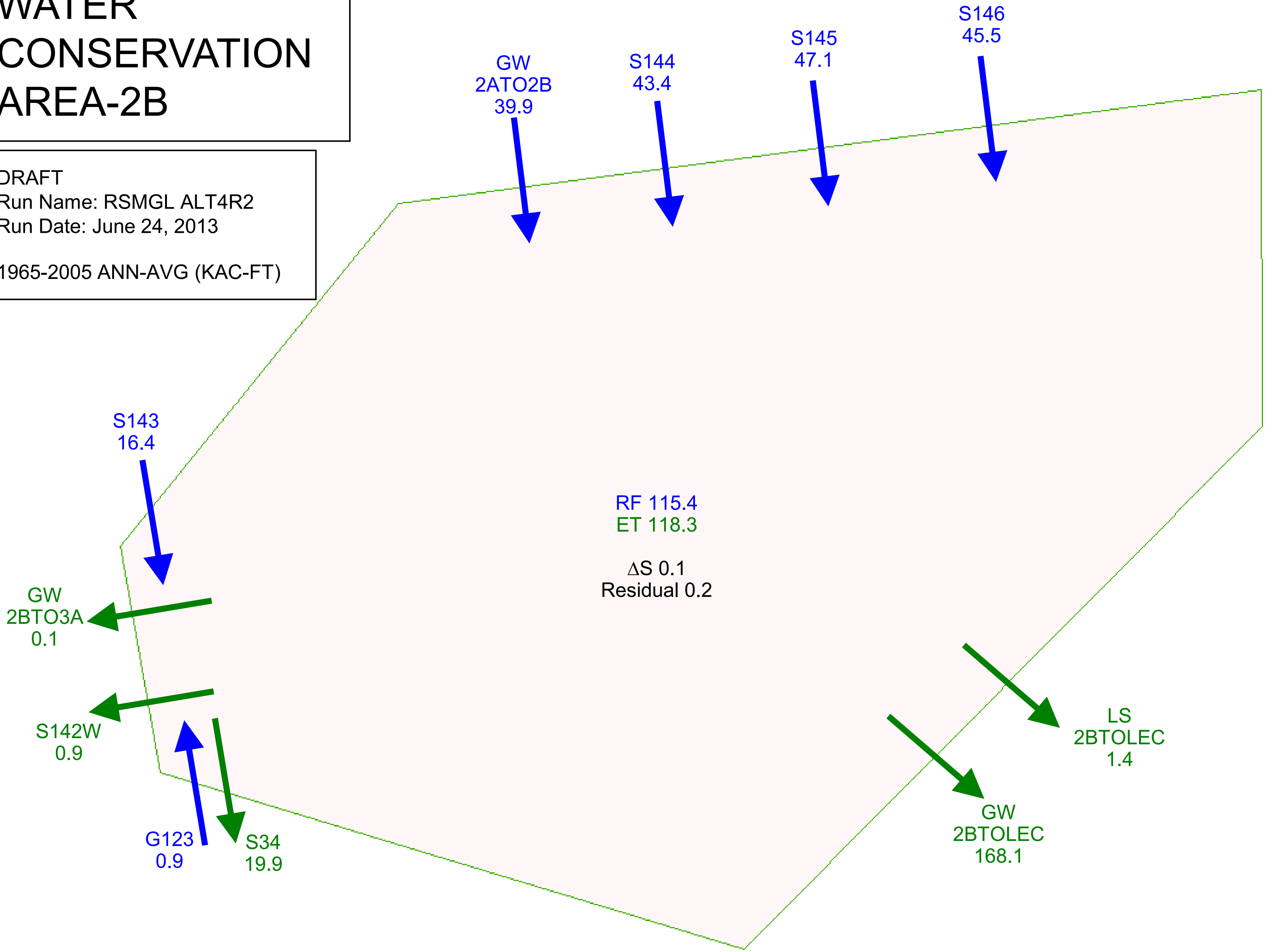
1965-2005 ANN-AVG (KAC-FT)

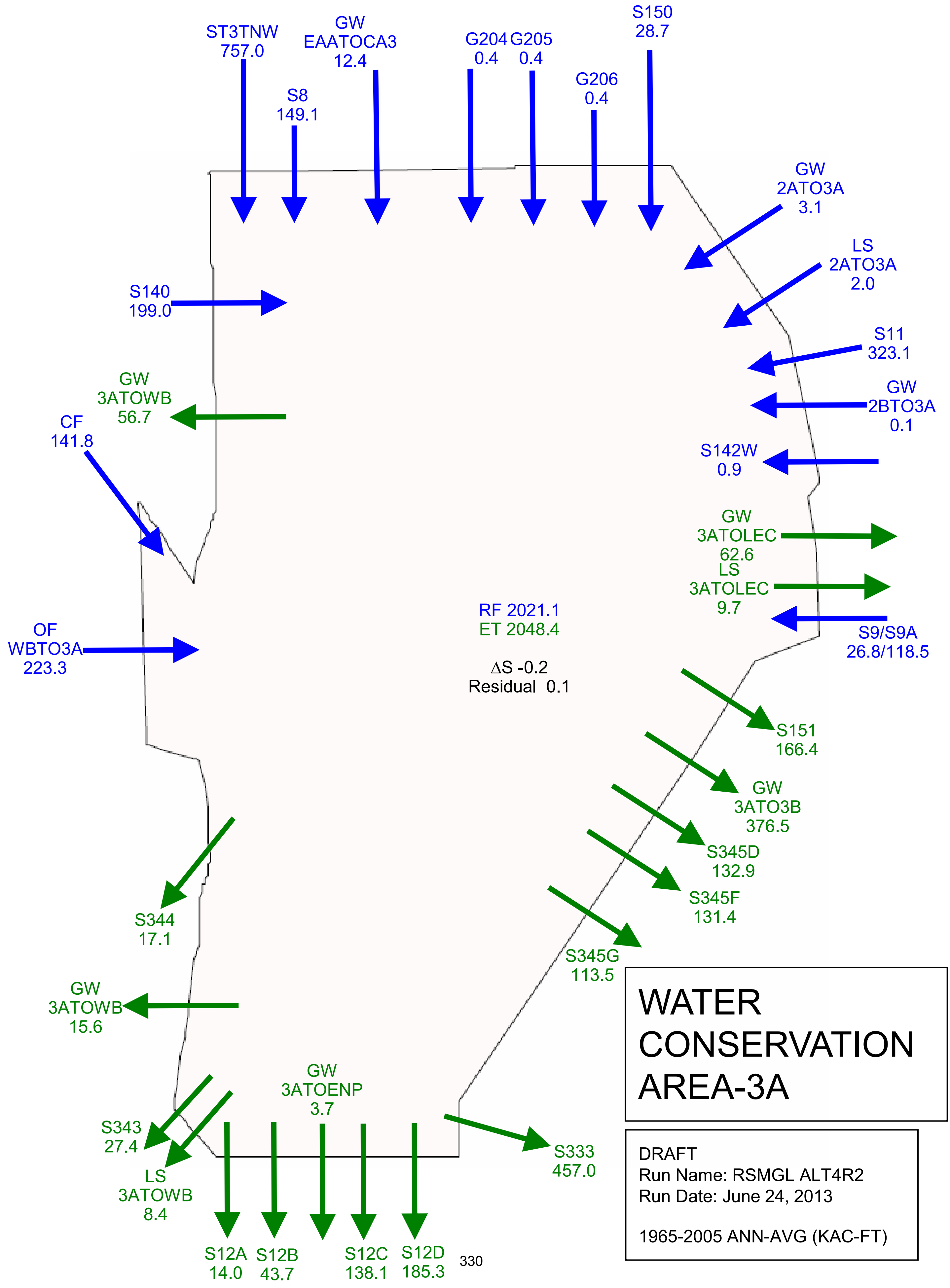


WATER CONSERVATION AREA-2B

DRAFT
Run Name: RSMGL ALT4R2
Run Date: June 24, 2013

1965-2005 ANN-AVG (KAC-FT)

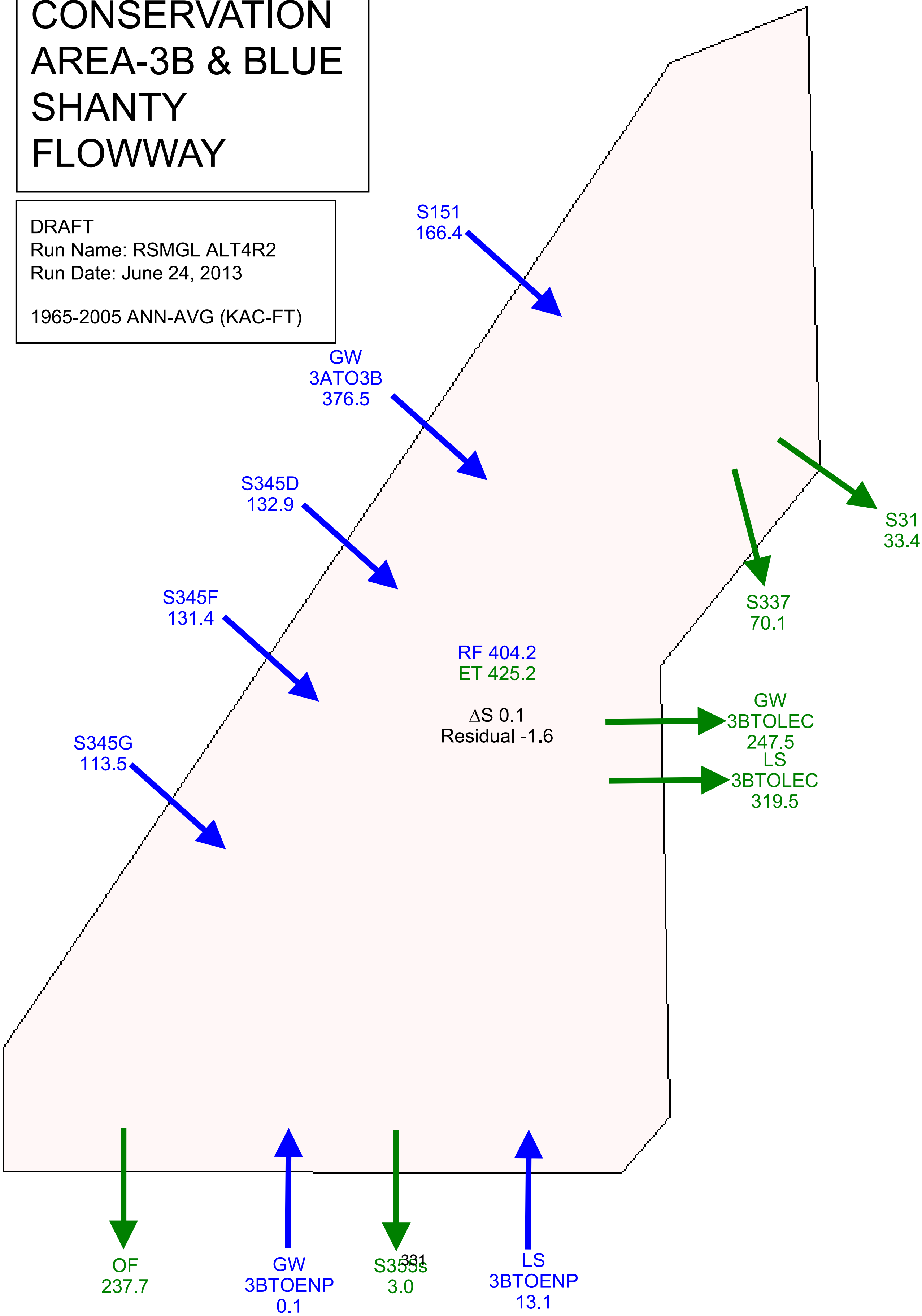



ST3TNW
757.0
S8
149.1
GW
EAATOCA3
12.4
G204
0.4
G205
0.4
G206
0.4
S150
28.7
GW
2ATO3A
3.1
LS
2ATO3A
2.0
S11
323.1
GW
2BTO3A
0.1
S142W
0.9
GW
3ATOLEC
62.6
LS
3ATOLEC
9.7
S9/S9A
26.8/118.5
S140
199.0
GW
3ATOWB
56.7
CF
141.8
OF
WBTO3A
223.3
RF 2021.1
ET 2048.4
ΔS -0.2
Residual 0.1
S151
166.4
GW
3ATO3B
376.5
S345D
132.9
S345F
131.4
S345G
113.5
S344
17.1
GW
3ATOWB
15.6
S343
27.4
LS
3ATOWB
8.4
GW
3ATOENP
3.7
S12A
14.0
S12B
43.7
S12C
138.1
S12D
185.3
S333
457.0
WATER
CONSERVATION
AREA-3A
DRAFT
Run Name: RSMGL ALT4R2
Run Date: June 24, 2013
1965-2005 ANN-AVG (KAC-FT)

WATER CONSERVATION AREA-3B & BLUE SHANTY FLOWWAY

DRAFT
Run Name: RSMGL ALT4R2
Run Date: June 24, 2013

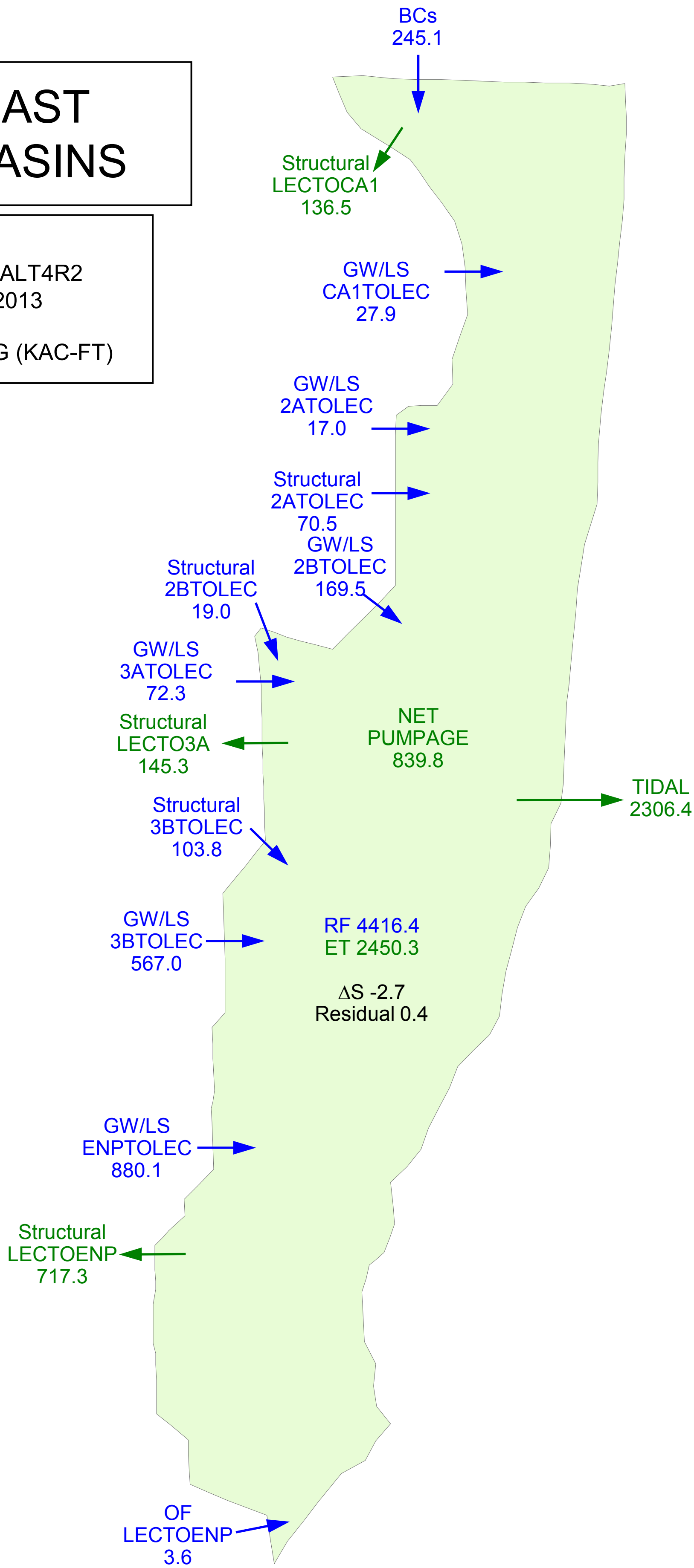
1965-2005 ANN-AVG (KAC-FT)



LOWER EAST COAST BASINS

DRAFT
Run Name: RSMGL ALT4R2
Run Date: June 24, 2013

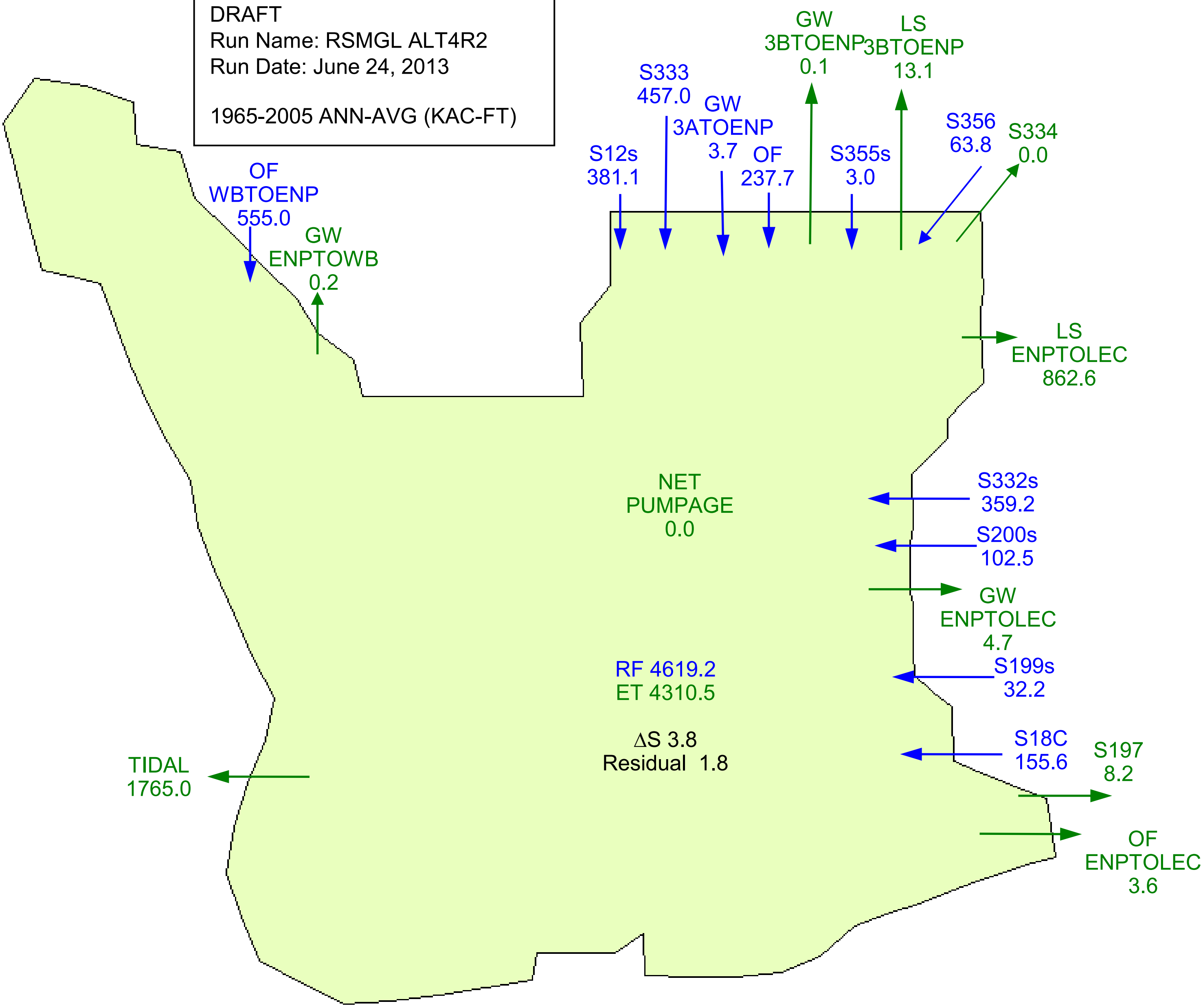
1965-2005 ANN-AVG (KAC-FT)



EVERGLADES NATIONAL PARK

DRAFT
Run Name: RSMGL ALT4R2
Run Date: June 24, 2013

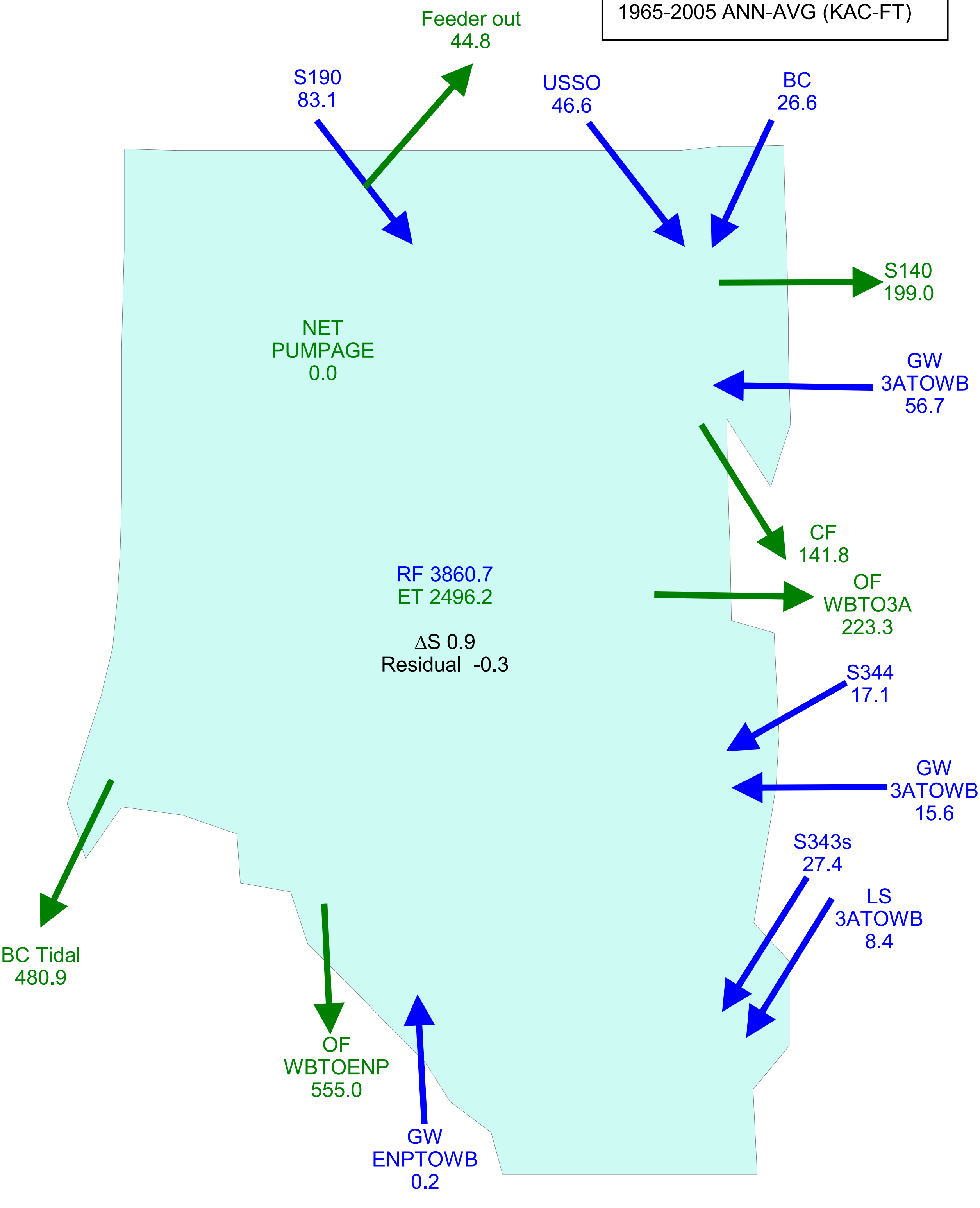
1965-2005 ANN-AVG (KAC-FT)



WESTERN BASINS

DRAFT
Run Name: RSMGL ALT4R2
Run Date: June 24, 2013

1965-2005 ANN-AVG (KAC-FT)



Note: Western Basins (WB) constitute L-28 Interceptor, Feeder Canal, L-28 Gap and East Collier Basins